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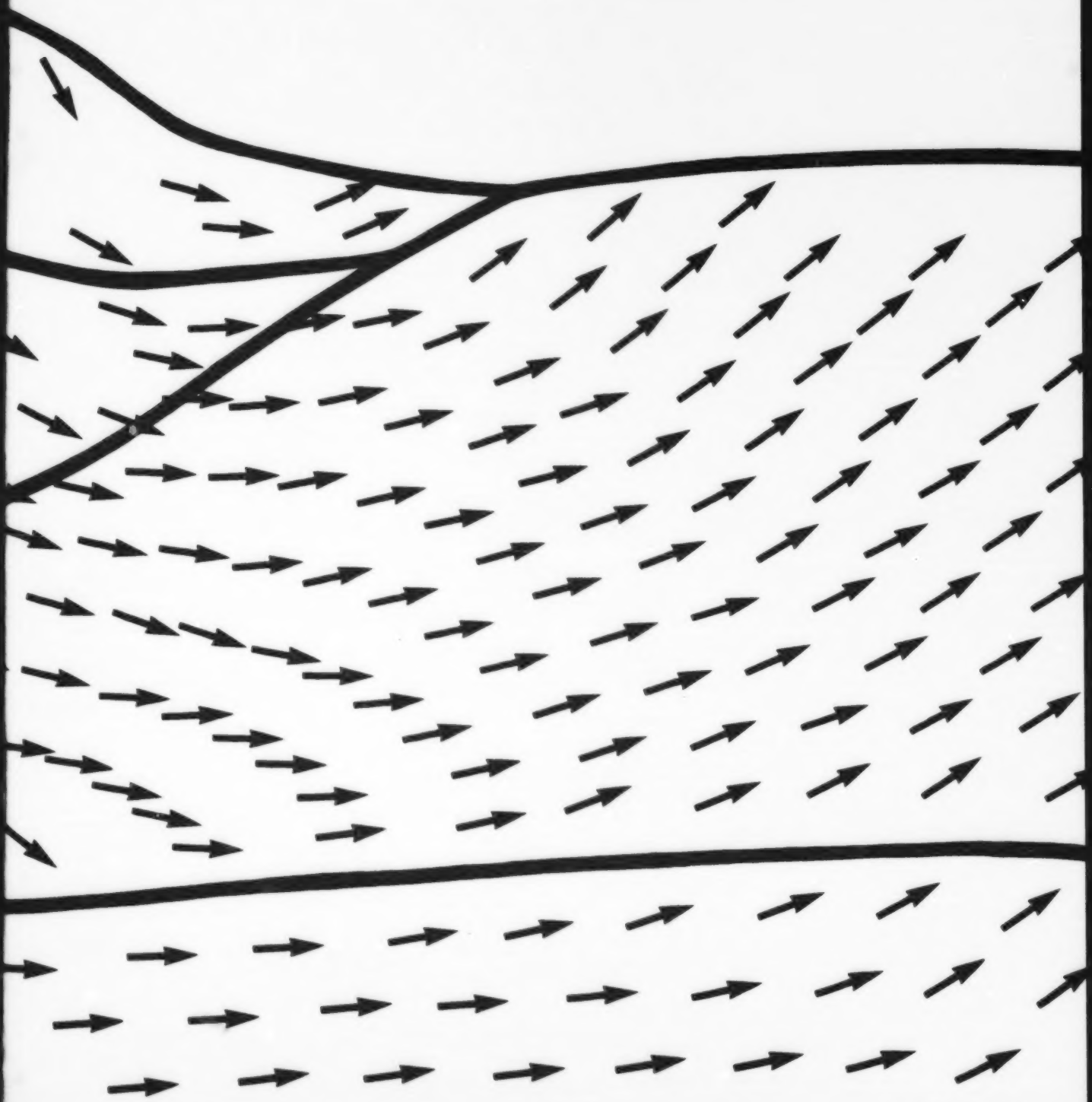
Geotextile Reinforcement in the Design of Low Embankments on Very Soft/Weak Soils



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Abstract:	The behaviour of low embankments on very compressible foundations is reviewed. The report discusses the limitations of current approaches for determining relevant soil and geotextile parameters, as well as the design of reinforced embankments. Also reported are the results of a series of laboratory tests to determine geotextile stress-strain characteristics, geotextile-soil interface properties, and the properties of fibrous peat. Recommendations are then made. Theoretical predictions are compared with observed embankment behaviour for a number of case histories and are shown to be in good agreement. These techniques are then used to examine the mechanisms governing the behaviour of geotextile reinforced embankments. Based on both theoretical studies and field observations, a series of recommendations are made regarding embankment analysis, the selection of appropriate geotextile reinforcement, and construction practice.
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1/ Introduction

1.1/ General

This report is concerned with the use of geotextiles as reinforcement for low embankments constructed on soft/weak foundations. These foundations may be subdivided into two groups, namely, fibrous peats and soft soils which behave as a purely cohesive material under rapid loading.

Fibrous peats are characterized by a high fibre content and water content, and a low ash content. They are highly compressible and permeable, particularly during the early stages of loading. Numerous embankments have been constructed over fibrous peat deposits with varying degrees of success. However, despite this extensive empirical experience there is still no simple, accepted design procedure for determining the stability and the likelihood of excessive shear distortions for embankments constructed on peat. The reason for this is the complexity of the foundation material itself. Firstly, at normal construction rates significant excess pore pressures will be developed. In general, these excess pore pressures will be far less than would be expected under undrained conditions. Thus the behaviour of a fibrous peat foundation cannot be categorized as truly drained or undrained. Secondly, the use of the field vane test for determining the shear strength of peat is of doubtful validity. Thirdly, because of the large deformations which occur during construction, the usual assumption of small strains implicit in conventional limit equilibrium analyses is not applicable for embankments on highly compressible peat deposits.

In order to make reasonable predictions of the stability and shear deformation of embankments on fibrous peat, it is necessary to have both an understanding of the behaviour of fibrous peats, particularly shear strength behaviour, as well as a method of analysis which permits consideration of construction pore pressures and large strains.

Soft cohesive soils are characterized by low to negligible fibre content, a much lower water content than fibrous peats, and a high ash content. At usual construction rates, they may be regarded as having a low permeability. Consequently, the end of construction settlement is relatively small (although subsequent consolidation and creep settlement may be quite large) and there will be relatively little excess pore pressure dissipation during construction. For these soils, the assumptions of small strains and undrained conditions implicit in conventional stability analyses are reasonable and considerable success has been experienced in using these simplified approaches to predict stability of unreinforced embankments constructed on these soils.

Geotextile reinforcement represents one alternative which may be considered by a designer wishing to increase embankment stability and reduce deformations. However, the inclusion of a geotextile into an embankment does complicate the analysis. If there is no deformation, there will be no force developed by the geotextile. In fact, the effect of the geotextile primarily depends on the stress-strain characteristics of the soil and the geotextile.

If the magnitude of the fabric force which can be mobilized at collapse is known, then limit equilibrium methods can be modified to incorporate the effect of the geotextile. The problem is determining the force which can be mobilized at collapse and this requires consideration of compatibility of deformations rather than limit equilibrium.

In developing recommendations for the use of geotextiles as reinforcement for low embankments on these soft/weak foundations, sophisticated finite element methods of analysis have been used to model the large deformations, the foundation material, the excess pore pressures, the embankment fill, and the reinforcement. To the extent possible, these techniques have been validated by comparison with documented performances of geotextile reinforced embankments. Good predictions of behaviour will require the use of these techniques together with a good knowledge of the geotechnical properties of the foundation itself. However, in most practical situations, time and funds are insufficient to perform an extensive evaluation of the foundation parameters and finite element analysis. These cases require a set of guidelines for the design and construction which can be used based on limited geotechnical data. The recommendations should indicate:

- the likely benefits arising from the use of geotextile reinforcement,
- what properties the reinforcing geotextile should have for a particular application, and
- how these properties should be determined.

This report is intended to provide these guidelines. If followed, it is expected that these guidelines will permit embankment construction on soft soils while avoiding collapse. However, geotextiles cannot eliminate consolidation settlement and large deformations may still occur. Some suggestions for estimating the magnitude of these settlements are included. The guidelines can be used with minimal soils data; however, "minimal" should be interpreted to include a good characterization of the nature and depth of the underlying deposit, including the identification of any soft "muds" which may be present beneath peat deposits.

A knowledge of the shear strength of any soft organic or inorganic soils is also essential. Finally, the possible implications of adverse geometry (e.g., a steeply sloping soft layer beneath the peat or a rapid change in the thickness of the peat) should be considered. Since no guidelines can cover all possible field situations, it is emphasized that they are only guidelines, and are intended to be used by qualified personnel. The information provided in this report should not be used without first securing competent geotechnical advice with respect to its suitability for any general or specific application.

1.2/ Scope

This study has been restricted to the use of geotextiles (including extruded geogrids) as a single layer of reinforcement in low embankments (2.5 m or less above original ground level) constructed on fibrous peat or very soft organic/inorganic soils. In the design and construction of these embankments, geotextiles may serve numerous other useful functions in terms of drainage, separation, and erosion control; however, these applications are beyond the scope of this report. Within this context, this study has involved:

a) an extensive review of the relevant literature;

b) a laboratory investigation of the effects of test parameters such as sample size and strain rate on the measured stress-strain characteristics of geotextiles;

c) a laboratory investigation into geotextile-soil interface shear strength for a number of geotextiles in contact with a natural sand fill and a lightweight fill (sawdust);

d) a laboratory investigation into the determination of relevant shear strength properties for a fibrous peat and the use of this information in predicting the performance of a geotextile reinforced embankment constructed on peat;

e) an examination of the behaviour of the two reasonably well documented field applications of geotextile reinforcement for embankments constructed on soft organic/inorganic soils;

f) a parametric study to determine the effect of geotextiles on the predicted performance of low embankments constructed on:
   - peat with, or without, a very soft underlying "mud" layer; and
   - very soft organic/inorganic soil;

g) development of guidelines which provide an indication of: -
   the likely benefits arising from the use of geotextile reinforcement;
   - the properties the reinforcing geotextile should have for a particular application;
   - how these properties should be determined;
   - the construction procedure and controls which should be adopted when using geotextile reinforcement.

2/ Performance of Embankments on Soft Foundations

2.1/ General

The problems associated with construction embankments on peat have long been recognized. In the 1950s and 1960s, considerable research effort was expended in trying to characterize the behaviour of peat and to develop techniques for constructing good quality roads over peat deposits. However, the amount of research into peat behaviour decreased significantly in the 1970s, despite the fact that many of the problems associated with design and construction of embankments on muskeg remained unresolved.

In this chapter, the literature regarding the design and performance of embankments constructed over peat deposits is reviewed, with particular regard to factors which affect the design and construction of these embankments using geotextile reinforcement.

2.2/ Terminology and Classification

The term "peat" is often used to include a vast range of organic soils which, as noted by Landva *et al.* (1983), may range from jelly-like, almost liquid "muds" whose organic content may not even originate from peat, to extremely coarse-fibrous meshes of woody remnants and fibres. The "mud" is commonly referred to as an amorphous-granular peat and the woody mesh as a fibrous peat. The most commonly used classification systems are the Radforth system (Radforth, 1969) and the Von Post system (as modified by Landva & Pheeney, 1980).

Landva *et al.* (1983) have proposed another classification which is primarily based on the ash content (this is summarized in Appendix I). In this classification, the range of ash content from 0% to 20% for peat (Pt) is based on a discontinuity in the water-content/ash content relationship. The introduction of a transition group, "peaty organic soils" (PtO), is based on the observation that organic soils with an ash content greater than 20% may still contain up to 50% fibres.

It may be anticipated that soils with a large fibre content will exhibit engineering behaviour different to soils with little or no fibre content. As a general observation, peats which have a high fibre content and low ash content will tend to be:

- more compressible (and hence will give larger settlements);
- more permeable (and hence "primary" settlement will occur more quickly);
- less prone to shear failure (because of the apparent cohesion due to the fibres, together with rapid increase in strength as excess pore pressure dissipates); and
- more likely to give large secondary settlements;

than organic soils which have a high mineral (ash) content and low fibre content.

2.3/ Relationship between Embankment Performance and the Underlying Deposit

Before discussing the performance of embankments on peat, it should be noted that even though an embankment may be stable, there will often be significant local shear failures within the peat and any underlying soft soil. Provided that these zones where the shear strength has been reached are contained or surrounded by soil which is not plastic (i.e., where the shear stress is less than the shear strength), collapse will not occur. However, the deformation of the embankment may be significantly affected by the development of these contained plastic regions.

Collapse of the embankment occurs when there is uncontained plastic flow. This may be characterized by the formation of a distinct sliding surface or by a situation where the shear deformations are such that continued addition of fill does not increase the height of the embankment unless some action is taken to stabilize the embankment. It should be emphasized that an embankment may be considered to have failed due to excessive settlement or differential settlement (which is a serviceability assessment) even though collapse, in the sense of uncontained plastic flow, has not occurred — i.e., an embankment that has collapsed has failed; but an embankment which is considered to have failed due to poor performance may not have actually collapsed.

The behaviour of a number of embankments constructed on peat has been reported in the literature and is summarized in Appendix J.

Muskeg areas are typically encountered in regions which have been subject to "recent" glaciation. Frequently, the depositional history of these deposits involves the sedimentation of clay, silts or lake marl, which may have some organic content, followed by the formation of a peat deposit which will usually be covered by some form of living vegetation. Because the peat has low unit weight, the clay, silt or marl in a recent normally consolidated deposit will be very weak just below the peat, although the increase in effective stress with increasing depth below the peat will generally result in a significant increase in strength with depth. For the cases reported in Appendix J, the vane shear strength in a soft stratum below the peat was typically in the range from 5 to 15 kPa, although shear strengths as low as 2.5 to 3 kPa have been reported (e.g., Lea & Brawner, 1963).

It has generally been observed (e.g., MacFarlane & Rutka, 1959; Lea and Brawner, 1963; Weber, 1969; Raymond, 1969) that construction and maintenance problems and shear failure are far more likely to occur when the peat is underlain by a weak soil than when it rests on a firm stratum. Consequently, these cases have been subdivided into two categories, namely, those underlain by weak soil (usually soft lake marl, clay or silt) and those underlain by much stronger soil (usually stiff clay, till or sand).

MacFarlane and Rutka (1959) reported very bad performance of a number of roads constructed on 4 to 6 m of peat overlaying 1.8 to 10 m of clay or silt. Lea and Brawner (1963), Weber (1969) and Raymond (1969) have reported the collapse of embankments (ranging in height from 0.5 to 2.5 m) constructed on peat (2 to 6 m thick) which was underlain by soft soil. Collapse in these cases usually involves formation of a crack in the embankment and a block-like slide moving on or just below the boundary between the peat and the soft soil. Recognizing that these failures have occurred, it should also be noted that embankments have been successfully constructed on similar deposits and that the embankment design and construction procedure can greatly affect behaviour of these embankments.

Embankments constructed on peat which is underlain by a firm foundation are less prone to collapse, although some cases have been reported. For example, Flaate and Rygg (1964), Ripley and Leonoff (1961) and Lupien (1983) report "shear failures" which, on the basis of the published data, do not appear to involve failure in the soil underlying the peat. These failures do not usually involve the formation of a definite sliding surface. Rather, the collapse involves rapid and excessive shear deformations which give rise to large embankment settlement, lateral movement and "mud" waves. In this context, collapse would be distinguished by the fact that additional fill will not increase the height of the embankment unless steps are taken to stabilize the embankment (i.e., allowing pore pressure dissipation during construction, use of berms, etc.).

2.4/ Rate of Loading and Pore Pressures

A review of the literature indicates that the rate of loading may significantly affect the performance of embankments on peat. Because of the high permeability of peat, some pore pressure dissipation will occur during construction; however, excess pore pressures will be developed and will generally reach a maximum value at the end of construction. The excess pore pressure beneath the embankment may be expressed in terms of the increase in total stress by:

$$\Delta u = \bar{B}\,\Delta\sigma_1 \qquad (2.1)$$

where: Δu is the excess pore water pressure
$\Delta\sigma_1$ is the increase in total stress
$\bar{B}$ is an empirical pore pressure parameter

(This approach is similar to that proposed by Tavenas and Leroueil (1980) for estimating excess pore pressures when significant dissipation occurs during construction of embankments on clay.)

Clearly, the maximum excess pore pressure (and hence $\bar{B}$) will depend upon the rate of loading and the drainage conditions. The range of $\bar{B}$ values that can be deduced from published field cases is typically 0.1 to 0.35.

Two very high values of $\bar{B}$ which have been reported warrant some comment. A value of 0.8 was obtained from the piezometer readings reported by Lupien *et al.* (1983). This data was for a 1 to 1.5 m high embankment constructed using a silty sand till with a uniformity coefficient of 10. This fill appears to have been relatively impermeable and significant excess pore pressures were measured within the fill itself. The peat was quite shallow (2.5 m) and was underlain by clay. For this case, a value of 0.8 does not seem unreasonable.

The value of $\bar{B}$ of 0.9 obtained from the results reported by Adams (1965) is more difficult to understand. Based on the published data, neither the peat nor the construction method appeared to be sufficiently different from other cases (including a nearby instrumented section where $\bar{B}$ was 0.15) to explain the high value of $\bar{B}$.

The values of $\bar{B}$ deduced from published cases provide some indication of the pore pressure which is likely to develop in peat, assuming that typical construction rates are adopted. However, these results should be used with caution. Usually, only one piezometer was used and hence the measured pore pressure may not necessarily be the maximum, although in most cases it appears that the piezometer was located in a position which would give a reasonably representative pore pressure. The variation in excess pore pressure with depth for the cases where more than one piezometer was used is shown in Figure 2.1. Although there is some scattering of the data, an "apparent end of construction" excess pore pressure "isochrone" can be constructed. Because of the paucity of data, a limiting isochrone has also been shown; this isochrone is more likely to provide a conservative estimate of the excess pore pressure distribution and is used in the design calculations in Chapter 4.

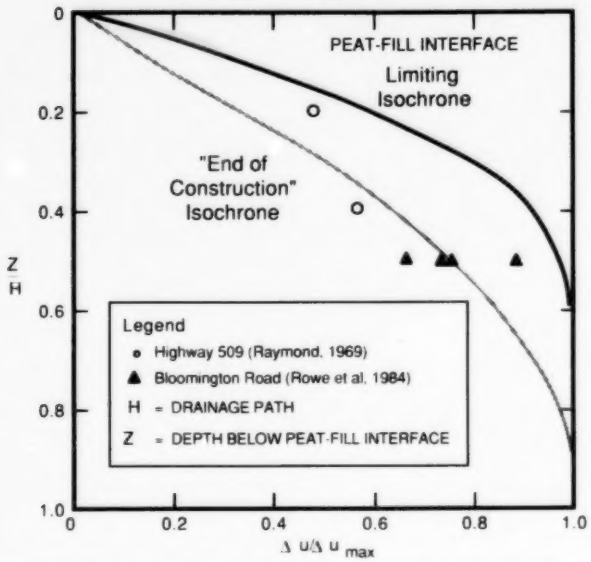
It may be expected that the magnitude of the maximum excess pore pressures, and hence the likelihood of failure, will depend on the rate of loading. Weber (1969) reported that in his experience with peat deposits in the San Francisco Bay area, fills in excess of 1.5 m above original ground level were subject to collapse. The exact height at which instability occurred depended upon the foundation condition and rate of loading. At a construction rate of 0.9 m of fill per week, he reported a collapse of a fill at a height of 2.4 m. At an even slower construction rate (0.15 to 0.3 m per week), fills could be constructed without failure to a height of 3 m.

Landva and La Rochelle (1983) have also noted that large shear deformations in peats with very low ash content are the result of a high degree of humification, a low degree of fibrosity, rapid loading causing high pore pressures, or any combination of these factors.

Both field and laboratory observations have established that embankments constructed on peat settle rapidly during construction (loading) and then continue to settle at a reduced rate after loading (e.g., Hanrahan, 1964; Adams, 1965; Weber, 1969; Rowe *et al.*, 1984). An examination of the time-settlement curves indicates that "primary" consolidation occurs relatively quickly (typically 5-200 days after construction) after which the settlement continues to increase approximately linearly with log time due to secondary compression. However, significant excess pore pressures have also been observed at times long after the time settlement curve indicates "primary" consolidation is complete.

These excess pore pressures appear to remain at a relatively constant value for considerable periods of time (e.g., Adams, 1965; Rowe *et al.*, 1984). The reasons for this behaviour have not generally been agreed upon (e.g., Adams, 1965; Landva and La Rochelle, 1983); however, it would seem reasonable to interpret these excess pore pressures as "creep induced pore pressures" which are the result of a time-dependent compression of both the peat fibres and of the organic structure itself. Clearly, the

Figure 2.1/ Variation in Excess Pore Pressure with Depth



possibility of the presence of these excess pore pressures for long periods of time may have important consequences when considering stage loading of a peat deposit. Unfortunately, the creep characteristics of a deposit are far more variable and difficult to predict than the short-term behaviour. The reason why embankments on some peat deposits generate creep pore pressure, while on other deposits they do not, has yet to be explained and much more research is required. In practical situations where stability is a problem (particularly where stage loading is being considered), it would be advisable to provide instrumentation to measure the pore pressure and to use this information to monitor the construction.

From the available data, it may be concluded that where drainage of the peat is provided and a slow rate of construction is used, there will be little difficulty in constructing low embankments (2 m or less) on peat, although settlements may be large. However, when an impervious fill is used or where the peat is underlain by soft marl or clay, it may not be possible to construct even low embankments on the peat unless special measures are taken to ensure stability (e.g., provide drainage, berms, geotextiles, use of lightweight fill).

2.5/ Secondary Compression and Surcharging

Several investigators (e.g., Lea and Brawner, 1963; Weber, 1969) have found that laboratory tests generally do not provide a good indication of secondary compression. The secondary compression observed in the field is quite variable and is particularly dependent on the load history of the deposit. Lea and Brawner (1963) have noted that this may partly explain the uneveness that develops on roads over peat that have not been preloaded.

Preloading has been found to be an effective means of reducing settlement due to secondary compression (Lea and Brawner, 1963; Weber, 1969; Samson and La Rochelle, 1972; Olivera, 1982). A method of estimating the magnitude and duration of the preload using laboratory data has been described by MacFarlane (1969); however, because of obvious differences between laboratory and field behaviour, this approach can only be considered a crude guide to design of the preload. It has typically been found that for low embankments on peats with depth up to 9 m, a temporary surcharge of 0.5 to 1.5 m will provide a significant improvement in embankment performance, provided that the surcharge does not cause a failure of the embankment. The reported duration of surcharging varies from a few weeks to as much as a year.

Since preloading can significantly affect the secondary settlement of an embankment, it is important to ensure that the entire length of the embankment over a soft deposit is surcharged. Surcharging only the deepest section of a deposit may give rise to a situation where the normally consolidated shallow sections of the deposit experience larger time-dependent settlements than the deeper sections which have been preloaded. This will give rise to an uneven road surface such as that observed on Bloomington Road (see Appendix C).

2.6/ Berms

Berms have been advocated as a means of ensuring the stability of embankments on peat. Brawner and Tessier (1969) recommend that the berms be about half the thickness of the main fill and about 6 m wide. While this recommendation is likely to be suitable for low embankments on soft deposits of less than 4 m, such berms may not be sufficient to ensure stability on deeper soft deposits.

Raymond (1969) has reported a number of cases where embankments were constructed using berms (on peat overlying a soft lake marl). In three cases, failure occurred despite the use of berms and in two of these cases the berm width was 7.5 m. On the basis of his investigation, Raymond recommended that:

> "In order to prevent large shear deformations within the peat-marl layers the berms should have a width 1.5-2 times the combined depth of the peat-marl layers .... Berms should preferably be built from their outer limits inwards."

To ensure stability, berms may have to be quite wide.

2.7/ Sawdust

When sawdust is available, its use as a lightweight fill provides a very effective means of reducing settlement and increasing embankment stability (e.g., Lea and Brawner, 1963; Flaate and Rygg, 1964; Raymond, 1969). It is well recognized that, to prevent decay, the sawdust must be placed such that it will be below the water table after short-term settlement has occurred. It has been found (Lea and Brawner, 1963; Nelson, 1974) that the density of the sawdust is relatively insensitive to moisture content and compactive effort and it can be adequately compacted by the passage of trucks. The sawdust is likely to be quite compressible during the early stages of loading but all the available evidence suggests that it has a quite high shear strength. (This is discussed in more detail in Chapter 3.)

2.8/ Sand Drains

Sand drains have been used on a number of projects (Brawner, 1958; Lake, 1961; Hillis and Brawner, 1961; Lea and Brawner, 1963; Weber, 1969). It was generally intended that these drains would increase the rate of settlement; however, field observation suggests that the sand drains did not increase the rate of settlement. Nevertheless, it has been found that sand-drain areas were more stable and subject to less vibration due to traffic than adjacent areas where no sand drains were used. It has been suggested that this is because the sand drains act as granular piles carrying loads to the firmer underlying soil.

2.9/ Geotextiles

Increasingly, geotextiles are being used to aid in the construction of embankments or weak compressive foundations. Examples have been reported in numerous articles (e.g., Bell *et al.*, 1977; Haliburton *et al.*, 1980; SCW, 1981; Barsvary *et al.*, 1982; Steward and Mohney, 1982; Hutchins, 1982; Olivera, 1982). In most cases, it is claimed that the geotextile significantly reduced the volume of fill required. Unfortunately, where test sections with and without fabric were reported, the soil profile beneath the test section was generally not the same and consequently a direct comparison of the fill required with and without geotextile is not justified. Nevertheless, there is some qualitative evidence to suggest that the geotextiles have improved embankment performance.

As a separator, the geotextile prevents the fill from being weakened by the intrusion of the soft foundation material. This is particularly desirable when the embankment is to be constructed on soft clay or highly humified or amorphous peat. As noted by Jarrett (1983), a separator may not be necessary for fibrous undecomposed peats since the fibres will compress and form a natural separation layer.

For fills constructed of free draining granular material, the contribution of a geotextile to drainage is of no consequence. However, for silty or cohesive fills, suitable geotextiles may be very useful in providing drainage at the top of the peat deposit thereby increasing stability by reducing excess pore pressures. For these fills, problems may also occur due to the rise of water in the fill to the fill surface as a result of capillarity. When this happens, pumping due to the passage of trucks may cause a loss of bearing capacity of the fill material itself. Lupien *et al.* (1983) have reported that geotextiles were particularly useful in stabilization and strengthening the fill under these circumstances.

When considering the role of geotextiles as reinforcement, two aspects need to be considered. Firstly, the geotextile may serve to bridge small local weak spots in the foundation thereby reducing fill indentation. Table 2.1 lists a number of fabrics which have been reported to serve this purpose reasonably well. Based on this empirical experience, it would appear that either woven or nonwoven geotextiles with grab tensile strengths in excess of 480 N are likely to reduce fill indentation into local weak spots in the peat.

Secondly, geotextiles may be used to provide general reinforcement of an embankment with the objective of reducing lateral spreading, increasing stability, and preventing tensile failure within any root mat beneath the embankments. This function is the major concern of this study.

A detailed study of three instrumented geotextile reinforced embankments constructed on weak or compressible soils is reported in Appendices A, B, C and D. The cases examined (summarized in Table 2.2) involve a very soft clay, a very soft peaty clay, and a highly compressible fibrous peat. In each case, field performance was compared with the behaviour calculated using the author's elasto-plastic finite element program (see Chapter 4 for a description of the program).

From examination of the embankment constructed cn very soft clay at Pinto Pass, it was concluded that:

- The outside-inside construction technique proposed by Haliburton *et al.* (1980) is superior to normal construction in horizontal lifts even without the use of fabric; however, the role of the fabric may also be enhanced by the technique, particularly for stiff fabrics. This finding was consistent with Raymond's (1969) recommendation that, when using berms, the berm should be constructed before the main embankment and that in constructing the

Table 2.1/ Geotextiles Used as Reinforcement

Reference	Fabric Name	Polymer	Type	Grab Tensile Strength (N)	Elongation at Break	Mass (g/m²)
Bell *et al.*, 1977	Fibretex	Polypropylene	Non-woven, needle punched, spun bonded	670	150	420
Haliburton *et al.*, 1980	Polyfilter X	Polypropylene	Woven, monofilament	1691	10	240
	Nicolon 66475	Polypropylene	Woven, multifilament	5560	18	730
	Nicolon 66186	Polypropylene	Woven, multifilament	1780	30	230
Barsvary & MacLean, 1982	Penroad 500	Polypropylene	Woven	800	22	150
	Terratrack T2415	Polypropylene	Woven	485	16	120
	Texel 7609	Polyester	Non-woven	480	80	280
	Typar 3401*	Polypropylene	Non-woven	600	62	136
	Permealiner 1195	Polypropylene	Woven, monofilament	1780	30	225
Lupien *et al.*,	?	Polyester	Non-woven	710	35	?

* *Also used by Hutchins (1982), Olivera (1982).*

berms, one should start at the outside and work towards the centre.

- The fabric has relatively little effect upon vertical settlements (provided collapse does not occur) but may significantly reduce lateral spreading.
- The fabric does increase the stability of the embankment. However, for fabric with low to moderate modulus, extremely large deformations may occur prior to the fabric reaching its tensile capacity, and in these cases, failure may be deemed to have occurred prior to rupture of the fabric.
- A knowledge of both the fabric modulus and tensile strength, determined under plane strain conditions, is considered to be essential for the analysis and design of geotextile reinforced embankments.
- Although some consideration must be given to the fill stiffness, it would appear that for the typical range of cases considered, a precise determination of the fill stiffness is unnecessary.

The analysis of the two embankments constructed on a very soft peaty-clay at Almere provides a direct comparison between unreinforced and reinforced embankment behaviour, as well as a comparison between the finite element analysis and a modified Bishop limit equilibrium analysis. Both the analysis and the field observations indicate the geotextile did significantly increase the stability of the embankment. In this particular case, where a high modulus fabric was used, failure of the reinforced embankment appears to have been governed by fabric pullout from the soil at a relatively low level of fabric strain (4-5%). Under these circumstances, both the finite element and limit equilibrium approach provided a good indication of the failure height of the embankment. It is likely that these methods would provide good predictions for other geotextile reinforced embankments under similar conditions involving failure of the fabric-soil interface at low strain levels. (This is discussed in more detail in Chapter 4.)

Appendices C and D describe the design, analysis, and field performance of two instrumented sections of a geotextile reinforced embankment on peat as part of Bloomington Road near Aurora. The embankment was designed and constructed by the Ministry of Transportation and Communications using the technology available at that time. The peat was highly compressible with average water contents of 445% and 785% at the two sections. The peat depth varied considerably with position (both longitudinally and laterally) having a maximum depth of 7.6 m. A moderate strength geotextile (Permealiner M1195) and high strength geotextile (Geolon 1250) were used to reinforce different sections of the embankment. The embankment was constructed in three stages over a period of one year.

Large settlements were observed at both instrumented sections, with a maximum fill thickness of 5.7 m and maximum settlement of 4.7 m being recorded. A significant component of the settlement appears to have been due to shear distortion, particularly during stage II construction. Tension cracks were observed in the fill (outside of the reinforced area) and in the peat during stage II construction. Large lateral movements were evident from "mud waves" and tree movements adjacent to the embankment. The embankment did not perform in accordance with the design.

Table 2.2/ Case Histories Analysed

Case	Appendix	Soil Conditions	Embankment Height*	Comments
Pinto Pass	A	3.6 m of very soft clay 2.5 kPa ≤ c_u ≤ 7.5 kPa overlaying stiffer clayey silty sand and stronger plastic clay.	2.4 m with 10:1 side slopes	The section analysed was successfully constructed using Geolon 1250 with $E_f \equiv 630$ kN/m.
Almere	B	3.3 m of organic clay $c_u \equiv 8$ kPa	(a) Collapse at 1.75 m	Unreinforced embankment.
		A 12 m deep ditch had been excavated at the toe of the embankment.	(b) Collapse at 2.75 m	Reinforced with Stabileaka 200 with $E_f > 1000$ kN/m. Failure due to fabric pullout.
Bloomington "A"	C&D	5.6 m of fibrous peat $\omega_n \equiv 445\%$	0.37 m Stage I 0.87 m Stage II	Settlement ≡ 1 m Settlement ≡ 3 m Reinforced with Permealiner M1195 with $E_f \equiv 150$ kN/m.
Bloomington "B"	C&D	6.8 m of fibrous peat $\omega_n \equiv 785\%$	0 Stage I 1.5 m Stage II	Settlement ≡ 3.1 m Settlement ≡ 4.6 m Reinforced with Geolon 1250 with $E_f \equiv 630$ kN/m.

** Above original ground level.*

Unfortunately, the devices used for measuring the geotextile strains failed prior to completion of the embankment and so the maximum fabric strains may not have been measured. However, these devices did show that:

- small geotextile strains were developed despite large settlements in stage I of construction, when the deformation was predominantly due to compression of the peat;
- large fabric strains (in excess of 21% for the Permealiner M1195) were developed in stage II of construction when large shear deformations were apparent;
- the transverse fabric strain varied appreciably with position; and
- the longitudinal fabric strains were significant.

The calculated (using a finite element analysis) and observed performance of the embankment constructed at Bloomington Road are compared in Appendix D, where it is shown that:

- The method of analysis adopted in this study provided good to excellent agreement between calculated and observed behaviour of both instrumented sections at the end of the two major stages of construction.
- In the analysis of embankment performance, the use of drained deformation and strength parameters determined from simple shear tests provided the best agreement between calculated and observed behaviour. (This is discussed in more detail in Chapter 3.)
- The use of field vane tests for estimating the shear strength of fibrous peat is highly questionable (see Chapter 3). Analyses performed using the vane strength profiles obtained at Bloomington generally did not provide a good agreement with observed behaviour.

The use of the remoulded vane strength in the analysis gave ridiculous results.

- The analysis indicates the role of the geotextile as reinforcement is highly dependent upon the amount of local shear failure which might occur in the absence of reinforcement. At low load levels (i.e., where there is a high factor of safety), the geotextile does not alter the embankment behaviour. As the extent of local plasticity increases, (i.e., the factor of safety is reduced), the reinforcement plays an increasingly more important role in reducing settlement and lateral movements.
- It should be emphasized that the major effect of geotextile reinforcement is to reduce lateral spreading and increase stability. Even a very high modulus geotextile will have relatively little effect on consolidation settlements.
- The fabric strains were greatest, and the geotextile effect most significant, during and just after construction of each stage. As the excess pore pressures dissipated and the peat gained strength, there was a decrease in fabric strain predicted in the analysis and observed in the field.
- The interface friction angle for the two geotextiles used at the Bloomington Road embankment was the same as that of the fill. In this case, interface slip was not a factor affecting the embankment's performance.

On the basis of the studies described in Appendices A to D and in the interim report for this project (Rowe *et al.*, 1982), it may be generally concluded that:

1) The single most important factor to be considered when selecting geotextiles for use as general reinforcement (i.e., to improve overall stability and embankment performance) is the fabric modulus. For a given situation, there is a limiting fabric modulus below which the geotextile has little or no effect as reinforcement. The magnitude of this lower threshold modulus will depend on the compressibility and strength of the fill and foundation material. On the basis of the current research, it is suggested that a minimum acceptable secant modulus over the strain range 0 to 5% is 125 kN/m. This will be sufficient to provide some reinforcement for embankments on very compressible soils (e.g., peat). There is also an upper limit beyond which any further increase in fabric modulus will not further improve the embankment performance. The measurement of modulus for a given fabric and the selection of a fabric for a particular situation is discussed in detail in Chapters 4 and 6, respectively.

2) For typical low granular fills (height, 2 m or less; 2:1 side slopes) and geotextiles where the interfacial friction angle is greater than 25°, interface slip is unlikely to occur for geotextiles with a modulus of 2000 kN/m or less. The likelihood of slip increases with increasing fabric modulus and under some circumstances will limit the effectiveness of geotextiles for cases where the modulus is well in excess of 2000 kN/m.

3) In situations where the geotextile-soil interface is cohesive (e.g., fabric on very soft clay), fabric-soil interface slip may control the stability of a reinforced embankment.

4) Geotextiles will not significantly reduce vertical consolidation settlement. For typical embankments on relatively shallow soft deposits (with depth less than the embankment width), an estimate of centreline consolidation settlement may be obtained assuming one-dimensional strain conditions.

5) Local yield within the soil mass will result in settlements greater than predicted assuming one-dimensional strain. The presence of a geotextile may increase embankment stability and reduce local yield with a consequent reduction in settlement. For cases in which the actual factor of safety without geotextile is greater than unity, the inclusion of a geotextile is unlikely to reduce vertical settlement by more than 20%. However, an appropriate geotextile may significantly reduce lateral spreading and increase the factor of safety.

6) The construction of embankments on peat deposits is often facilitated by the presence of a root mat overlying the deposit. Provided failure does not occur, this root mat will often be sufficiently stiff to govern the embankment performance and only a very high modulus geotextile would directly influence embankment response. However, increasing embankment height tends to cause tensile failure within the root mat. The presence of a geotextile of even moderate modulus will reduce the likelihood of the root mat splitting beneath the embankment. Under these circumstances, the primary function of the geotextile is to maintain the integrity of the root mat. Again, the modulus of the fabric is the major

characteristic governing the effectiveness of the geotextile for this purpose. Increasing fabric modulus reduces root mat tension near the embankment centreline but may in fact increase root mat tension outside the reinforced zone, causing tension cracks.

Although it's well recognized that a root mat may significantly increase stability and reduce settlement, there are many practical problems associated with establishing the depth and characteristics of a root mat. In particular, the beneficial effects of the root mat should be seriously considered only if the designer is convinced of the general competence of the root mat, since any localized weak spots may negate these benefits.

3/ Soil Parameters for Use in the Design of Reinforced Embankments

3.1/ General

The performance of a geotextile-reinforced embankment will depend upon the properties of the fill, the fabric, the fill-fabric interface, and the underlying soil. In this section, consideration is given to the selection of soil parameters. The fabric and fill-fabric interface properties are discussed in Chapter 5.

3.2/ Sand Fill Properties

The stiffness of a granular material may be approximately represented in terms of a Young's modulus E which is a function of stress level. Several investigators have related Young's modulus, E, to the minor principal stress, by Janbu's equation, viz:

$$\left(\frac{E}{p_a}\right) = K\left(\frac{\sigma_3}{p_a}\right)^n \qquad (3.1)$$

where typical values of K and n are given in Table 3.I (the atmospheric pressure p_a is used to obtain a dimensionless equation). In general, the tests reported in Table 3.1 were performed at minor principal stress levels much higher than those typically expected in low embankments.

A series of triaxial tests were performed to determine the stiffness of the fill material from Bloomington Road. A typical grading curve for this material is shown in Figure 3.1. The tests were performed at two densities:

- loose: γ_{dry} = 17 kN/m³; and
- dense: γ_{dry} = 19.4 kN/m³

over a range of low confining stresses (σ_3) representative of the anticipated field stresses. At these low stress levels, particular care was taken to prevent the membrane from influencing the results.

The parameters K and n are most conveniently determined by transforming Equation 3.1 into the form:

$$\ln\left(\frac{E}{p_a}\right) = \ln K + n \ln\left(\frac{\sigma_3}{p_a}\right) \qquad (3.2)$$

and plotting the logarithm of dimensionless Young's modulus versus the logarithm of dimensionless cell pressure as shown in Figure 3.2. The values of K and n may then be determined from the intercept and slope of the line of best fit through the data points. It can be seen that Equation 3.2 provides a very good description of fill behaviour (the deviation of three points from these lines can be largely explained by a variation in density from the mean value of the test samples).

Table 3.1/ Stress-Strain Characteristics of Granular Fills

No.	Description	Source	ϕ	K	n	υ
1.	Antelope Valley Sand	Lade (1980)	36.7°	450	0.38	0.3
2.	Silt (Cannonsville Dam)	Hirshfield & Poulos (1963)	--	360	0.56	--
3.	Loose uniform sand	Duncan & Chang (1970)	30.4°	295	0.65	--
4.	Dense uniform sand	Duncan & Chang (1970)	36.5°	2000	0.54	--
5.	Sand	Duncan & Chang (1970)	39.0°	720	0.6	0.2
6.	Sand	Vesic & Clough (1968)	35.5°	300	0.83	0.35
7.	Uniform sand	Rowe (1978)	32.0°	440	0.88	--
8.	Well graded sand	Kulhawy, Duncan & Seed (1969)	50.0°	300	0.5	--
9.	Poorly graded sand	Kulhawy, Duncan & Seed (1969)	40.0°	1200	0.5	--
10.	Silty sand	This study				
	Loose		32.0°	57	0.51	0.35
	Dense		38.0°	220	0.53	0.35

The values of K and n determined from these tests are summarized in Table 3.1. The value of n is typical of that previously reported in the literature. The value of K is lower than commonly reported. The parameters K and n, and the friction angle ϕ are sensitive to the angularity of the sand, the grading and the density of the sand, and these factors may partly explain the differences apparent in Table 3.1. However, it is considered that the very low confining stresses used in these tests was also a major factor responsible for the low values of K. This implies that Equation 3.1 is applicable over a limited stress range and that it should not be used to extrapolate modulus values for confining stresses outside the range over which K and n were determined.

As noted above, the density of the fill may have a significant effect upon the strength and stiffness of the fill. This, in turn, will depend on the level of compaction which can be achieved. When constructing embankments on soft foundations it is particularly difficult to obtain good compaction of the first 1 to 1.5 m of fill. Consequently, it is suggested that the fill strength and stiffness parameters used in the analysis and design of low embankments on these soft foundations should correspond to the loose state. This will provide conservative results. The use of a geotextile may improve the level of compaction which can be achieved for fill on a soft foundation, thereby increasing the strength and stiffness of the fill. However, at present, insufficient data are available to allow an assessment of any improvement in embankment performance due to the improved compaction resulting from the use of geotextiles. Additional research is required to investigate this aspect of reinforced embankment behaviour. Until that research is completed, it would seem prudent to adopt parameters for a loose fill even though geotextile reinforcement is used.

Figure 3.1/ Grain Size Distribution — Natural Fill Material

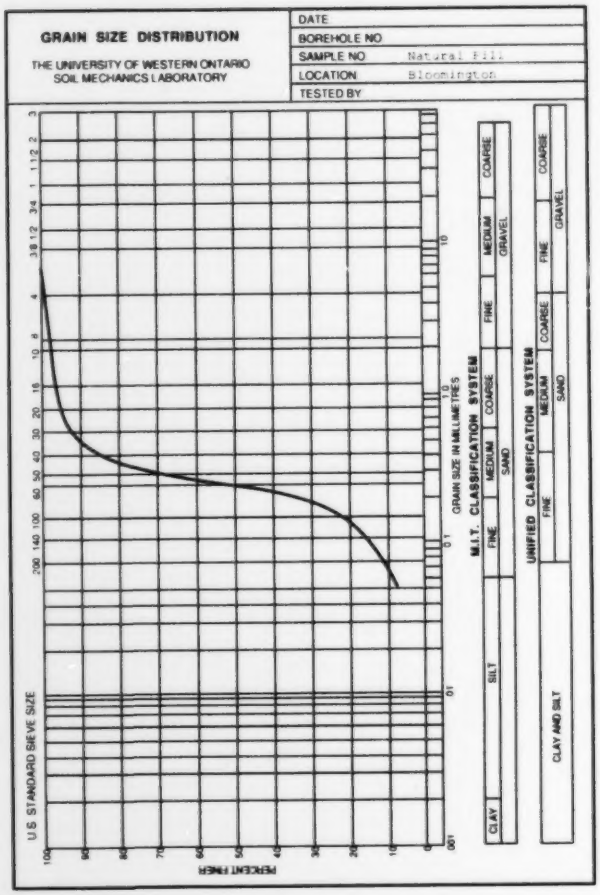


Figure 3.2/ Dimensionless Modulus vs. Dimensionless Cell Pressure (loose and dense samples)

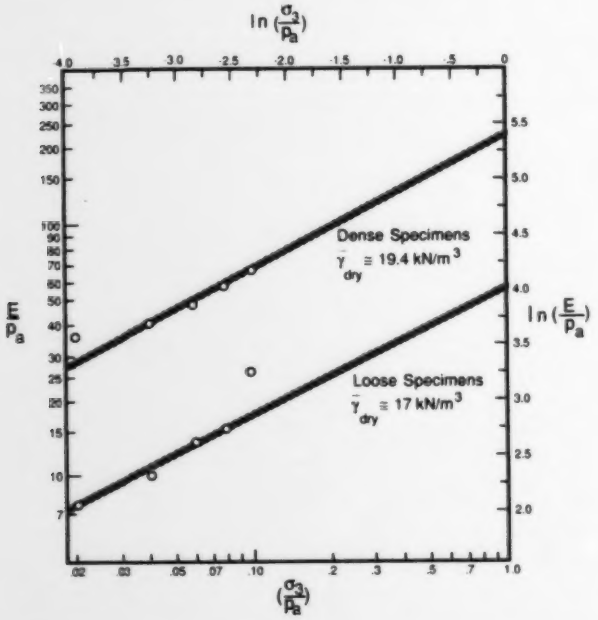
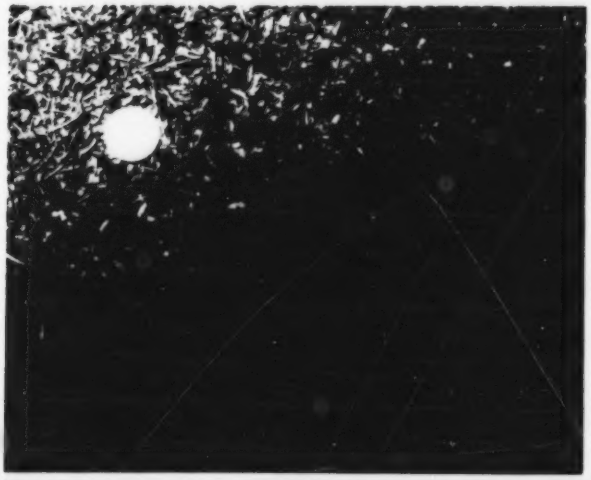


Table 3.2/ Strength Parameters for Sawdust

c' (kPa)	ϕ'	Comments	Source
14.5	27°	Drained triaxial tests.	Jackson (1980)
--	36°-39°	Dutchcone penetrometer	Jackson (1980)
--	>27.5°	Drained triaxial at 30%	Smith *et al.* (1980)
--	31°	Drained triaxial on "hog fuel"	Nelson *et al.* (1974)
--	50°	Drained direct shear tests. Definition of failure not given.	Lea & Brawner (1963)
--	>37°	Drained direct shear tests at 10% strain. Not at failure.	Present study

Figure 3.3a/ Sawdust as Used in Direct Shear Tests

Figure 3.3b/ Sawdust as Used in Triaxial Tests (large particles removed)

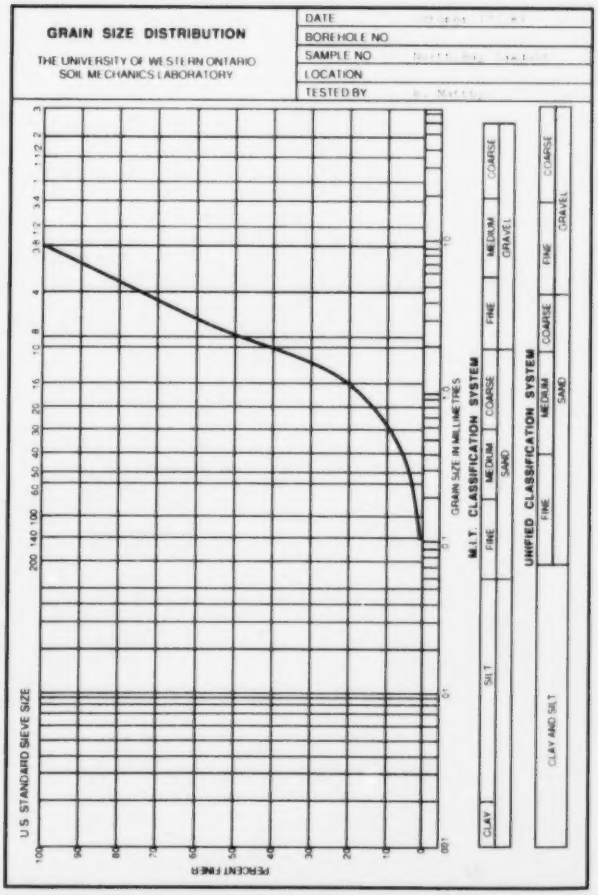


3.3/ Sawdust as a Lightweight Fill

Sawdust has found wide acceptance as a lightweight fill for use in the construction of embankments over very compressible foundations; however, relatively little has been published concerning the engineering properties of sawdust. Table 3.2 summarizes the available shear strength data for sawdust. From the published data, it is evident that the sawdusts which have been studied exhibit relatively high friction angles and large strains prior to failure. The deformation parameters for sawdust do not appear to have been studied.

As part of this study, a series of CID triaxial tests and direct shear tests was conducted on sawdust provided by MTC. This sawdust is to be used for a road embankment to be constructed over peat on Highway 575 in the North Bay area. The sawdust is shown in Figure 3.3. For the triaxial tests, the sawdust was passed through a #4 sieve to separate the larger woodchips that are evident in Figure 3.3. The grain size distribution of the sawdust is shown in Figure 3.4. The triaxial tests were conducted on 50 x 110 mm samples consolidated to low confining stresses (typically less than 15 kPa) and with an average saturated unit weight of 10.5 kN/m^3. Lea and Brawner (1963) have noted that neither compactive effort nor moisture content are critical when placing sawdust fill. Nelson *et al.* (1974) have also shown that the unit dry weight of sawdust is relatively insensitive to moisture content and that sawdust was relatively insensitive to compactive effort.

Figure 3.4/ Grain Size Distribution of Sawdust



Consequently, it is considered that the density of samples examined in the laboratory tests are likely to be reasonably representative of field conditions. To minimize aerobic deterioration, the sawdust was saturated and kept under water in a refrigerated room at 6°C prior to testing.

The stress strain curves obtained from four CID triaxial tests are shown in Figure 3.5. In each case, Skempton's B parameter was in the range from 0.96 to 0.99 prior to testing. It is evident from Figure 3.5 that failure had not occurred at strains of up to 50%. There was no evident effect of consolidation pressure over the range considered (5 to 12 kPa). The difference between the curves primarily reflects minor sample variations between tests. The initial drained modulus, E_i, at strains less than 1% are given in Table 3.3. The average initial modulus was 1.3 MPa. There was a decrease in sample stiffness for strains between 1% and 5%, at which point the tangent modulus, E_t, assumed a relatively constant value of approximately 0.6 MPa. Poisson's ratio was negligibly small in each test. At strains of less than 5%, Poisson's ratio, υ, was less than 0.01. Even at axial strains approaching 50%, the ratio of lateral to vertical strain was only 0.11 (this value takes account of large strain effects). The very low apparent Poisson's ratio reflects the very compressible nature of the sawdust particles.

Since the sawdust had not failed at termination of the tests at an axial strain of 50%, the shear strength cannot be determined. However, it is apparent that the sawdust does not behave like conventional soil due to the highly compressible nature of the sawdust skeleton. At large strains, the geometry of the triaxial specimens precluded the formation of a failure plane.

Direct shear tests were also performed using a 152 x 152 mm shear box. Again, the sawdust could not be brought to failure before the travel limit of the apparatus was reached. Based on an envelope to the stresses after 14 mm displacement, as shown in Figure 3.6, it would appear that the friction angle exceeds 37°. The high strength in the direct shear tests is due to interlock of the randomly oriented particles across the potential failure plane. Complete failure would require that these wood particles tear out of the sawdust matrix outside of the failure plane or else the wood particles themselves must shear.

Additional research is required to fully understand the geotechnical characteristics of sawdust. On the basis of available information, it would appear that sawdust such as that tested in this study has a friction angle at least equivalent to that of loose sandy fill. The modulus of the sawdust varies between 1.3 MPa and 0.6 MPa, depending on stress level.

Table 3.3/ Modulus Values for Sawdust

Test #	σ'_c (kPa)	γ (kN/m³)	E_i (MPa)	E_t (MPa)
1	5	10.4	1.38	0.58
2	12	10.6	1.10	0.57
3	5	10.8	1.35	0.64
4	10	10.4	1.68	0.60

Figure 3.5/ Stress-Strain Curve for Sawdust — CID Triaxial Tests

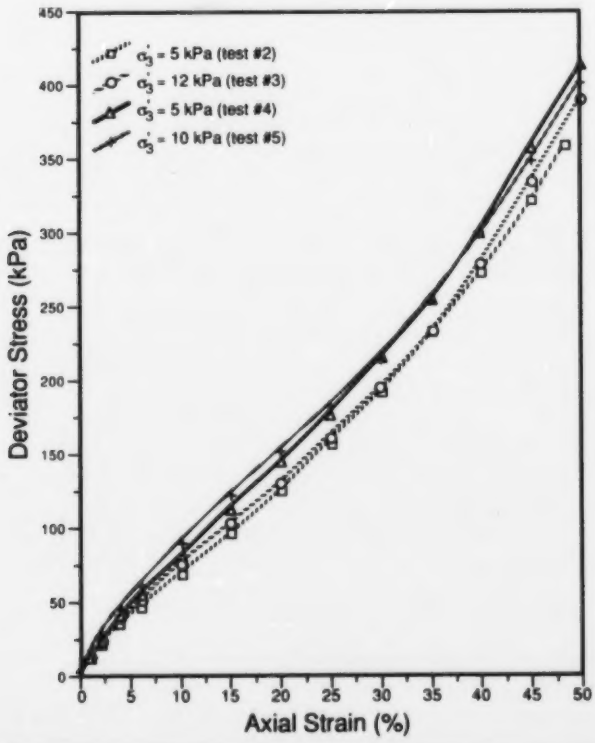
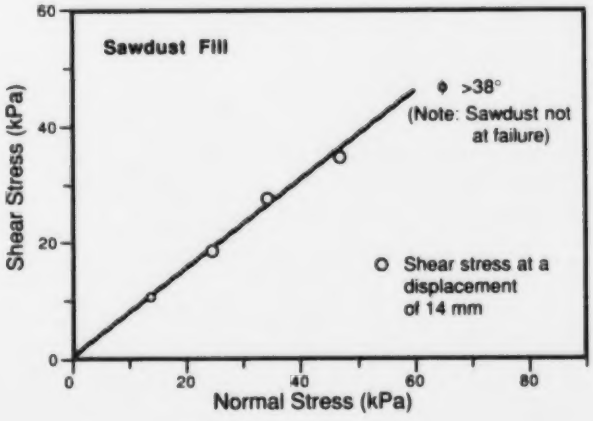


Figure 3.6/ Shear Stress vs Normal Stress — Sawdust Fill



3.4/ Settlement and Deformation Parameters for Peat

Peat deposits are rarely uniform. Generally, there will be variations in the type of peat, the degree of decomposition (humification) and the moisture content with both depth and lateral position beneath an embankment. These factors will all affect the compressibility of the peat and the consequent embankment settlements. Nevertheless, experience (e.g., Lea and Brawner, 1963; Weber, 1969; Samson and La Rochelle, 1972) has shown that reasonable predictions of field settlement can be made on the basis of laboratory consolidation data, provided that:

- parameters are obtained for representative samples of the peat;
- the depth of peat is known; and
- excess shear deformations do not occur (this will depend on the shear strength of the peat, the height of fill, and the rate of loading).

By relating compressibility to peat classification, the amount of testing required at a particular site can be minimized. Provided that lightweight fill is not being used, it may often be sufficient to design for the typical and worst sections only. If lightweight fill is used, then more care is needed in estimating settlements to ensure that the lightweight fill is submerged after construction. Shear deformations can be minimized by appropriate design and good construction procedure as discussed in Chapter 4.

The compressibility of the peat may be determined from conventional one-dimensional consolidation tests using *multi-stage loading* (e.g., 3, 6, 12, 24, 48 ... kPa) with load increments being applied every 24 hours. The effects of loading (multi-stage versus single stage) and sample disturbance upon the results from consolidation tests has been discussed by several investigators (e.g., Samson and La Rochelle, 1972; Landva and La Rochelle, 1983). With regard to the use of laboratory values for predicting embankment deformations, the key findings were:

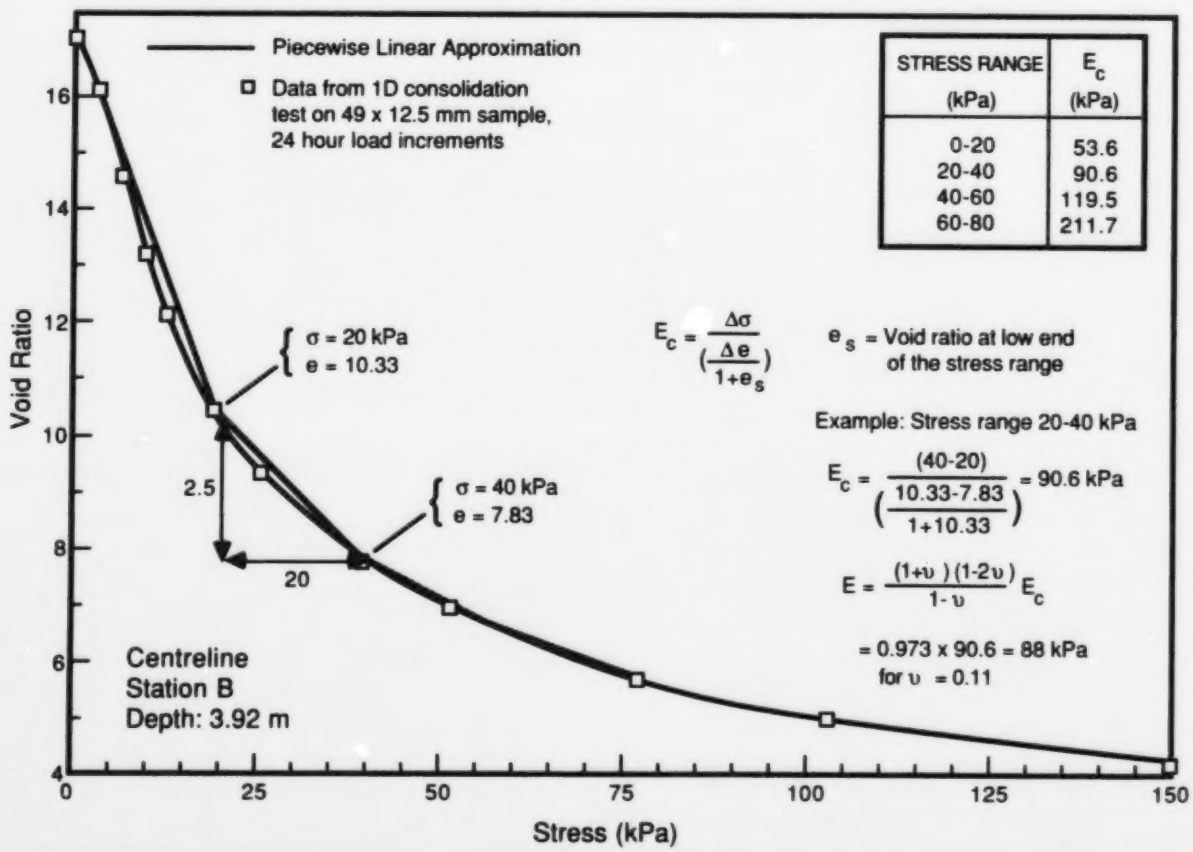
1) multi-stage and single stage loading led to the same amount of compression, and

2) disturbance had no significant effect upon the compression of the peats tested.

The results from the 24 hour multi-stage test indicated that the primary consolidation occurring in one given load increment is decreased by the amount of volume change which took place during the secondary compression of the previous load increment.

Additional evidence in support of these findings comes from the good agreement between calculated and observed settlements of embankments on peat reported by Weber (1969) and Rowe *et al.* (1984, and Appendix D). In both cases the deformation parameters were determined using 24 hour tests on samples obtained using 50 mm diameter Shelby tubes.

Figure 3.7/ Typical Void Ratio-Stress Curve for Peat



One approach to predicting the vertical compression, ε_z, of peat is to use the equation:

$$\varepsilon_z = \frac{C_c}{1+e_o} \log_{10} (\sigma_o' + \Delta\sigma) / \log \sigma_o' \qquad (3.3)$$

in which: C_c = compression index
e_o = initial void ratio
σ_o' = initial effective overburden pressure, and
$\Delta\sigma$ = change in effective stress due to embankment construction.

However, this approach has many associated difficulties (Landva and La Rochelle, 1983). For example, this equation is particularly sensitive to errors in the determination σ_o' and it neglects the effects of any preconsolidation of the peat. (The equation could, of course, be modified to include preconsolidation pressure but this creates additional problems because this equation will also be very sensitive to errors in determining the preconsolidation pressure and recompression index, both of which are difficult to determine accurately for peat.)

A better simple approach for predicting vertical compression of peat is to plot the data from the consolidation tests on a linear plot as shown in Figure 3.7 and to use these data in the equation:

$$\varepsilon_z = \frac{\Delta e}{1+e_o} \qquad (3.4)$$

where: Δe = the change in void ratio measured from the plot for the change in stress from σ_o' to $\sigma_o' + \Delta\sigma$;
and e_o = the initial void ratio of the peat.

This equation is not particularly sensitive to the uncertainty regarding the initial overburden pressure σ' (which usually will be quite small) and reasonable predictions can probably be obtained even if σ_o' is taken to be zero.

The initial overburden pressure may be estimated from a knowledge of the unit weight of the peat and the position of the water table. The water table is usually at or very close to the ground surface. Based on a survey of published data, the typical range of unit weight for peat is from 8.6 to 11.8 kN/m^3 with an average value of about 10 kN/m^3.

For more sophisticated analyses (e.g., finite element analysis), it is necessary to determine the modulus of the peat over different stress ranges as well as Poisson's ratio. Poisson's ratio, which has been determined by several investigators (see Table 3.4), is generally quite low. In the absence of test data, 0.15 is considered to be a reasonable value to adopt.

"Young's modulus" may be determined from triaxial tests over the appropriate stress ranges and may be calculated from the results of one-dimensional consolidation tests, provided that Poisson's ratio can be estimated. (In fact, the modulus is not very sensitive to Poisson's ratio for typical values of this parameter.) To illustrate the procedure, the modulus is calculated from consolidation data for the stress range 20 to 40 kPa in Figure 3.7 using a Poisson's ratio of 0.11 (this value was determined from triaxial tests on the peat). These data were obtained as part of an investigation of the Bloomington Road embankment. Similar calculations were made for a range of depths and stress levels and the results are shown in Figure 3.8.

As with many other peat deposits, the nature of the peat at Bloomington Road varied with both position and depth, ranging from a very fibrous peat with low humification and a moisture content up to 1040% to a highly humified, almost amorphous peat with water contents as low as 160% (see Appendices C and F for more details). This variability is apparent in the variation of the modulus values determined from consolidation tests. However, it is also apparent that a reasonable envelope of "Young's modulus" variation with depth (which reflects the compressibility of the peat) can be established for each stress range as shown in Figure 3.8.

Typical confined modulus values for peat lie in the range from 60 to 300 kPa. A value of 125 kPa would be considered representative, although clearly field values will often be greater and will occasionally be less than this value.

Table 3.4/ Poisson's Ratio

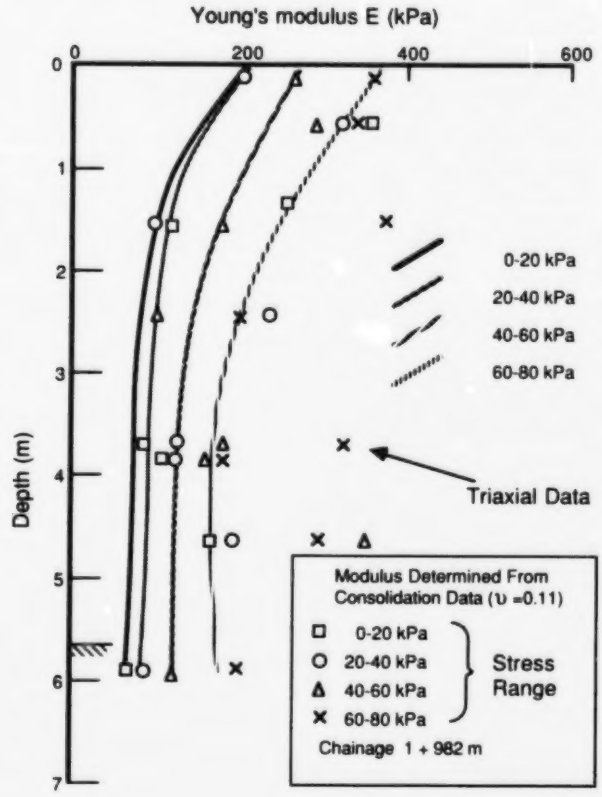
Case	ω_n (%)	Classification	Poisson's Ratio
Adams (1961)1	375-430	Woody fine-fibrous Categories[2] 9-11	0.13-0.33
Hollingshead & Raymond (1972)	560	Fine fibrous-amorphous	0.24-0.36
Landva (1980)	1200-1500	Sphagnum peat 85-98% leaves 10-15% fibrous $H_{3-5}\ B_4\ F_{1-2}\ R_{12}$[3]	0.1
This study (Appendix E)	400-1315	Predominantly fine fibrous $H_{5-7}\ B_3\ F_2\ R_{1-2}\ W_1$[3]	0.08-0.21

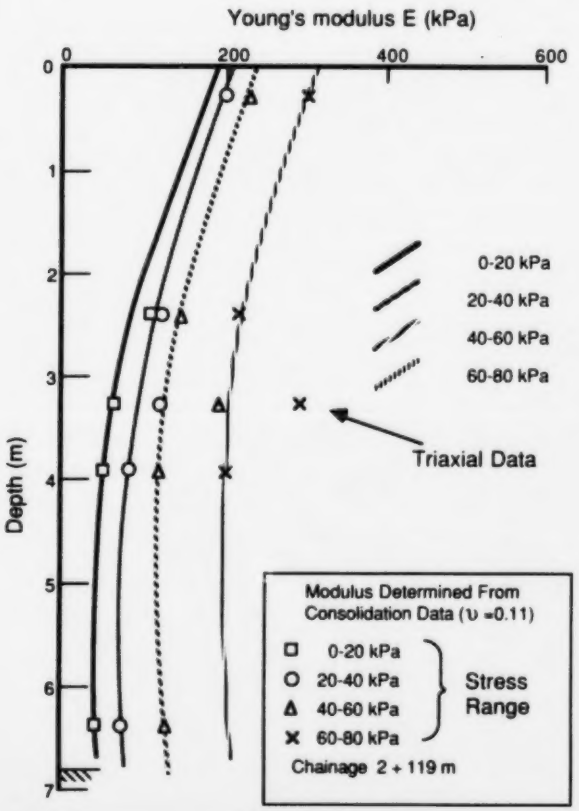
1 *Deduced from a zero lateral strain triaxial test using the relationship* $\upsilon = \frac{K_o}{(1+K_o)}$

2 *See Table 2.1*

3 *See Table 2.2*

Figure 3.8/ Variation in Modulus with Depth as Deduced from ID Consolidation Tests and CID Triaxial Tests





The coefficient of earth pressure at rest, K_o, for peat may be expected to be less than or equal to 0.3 (Adams, 1965; Gautschi, 1965; Landva, 1980). In view of the low values of Poisson's ratio for peat, it may also be expected that any increase in horizontal stress due to vertical loading will also be small (perhaps as low as $0.1 \cdot \Delta\sigma'$), at least until local shear failure occurs at that particular location in the peat.

3.5/ Secondary Compression

Numerous investigators have attempted to relate secondary compression, ε_t, which will occur in the field between time t_o and t to a laboratory-measured coefficient of a secondary compression C_s by means of an equation of the form:

$$\varepsilon_t = C_s \log_{10} (t/t_o) \qquad (3.5)$$

(e.g., Adams, 1965; MacFarlane, 1969; Weber, 1969; Samson and La Rochelle, 1972). However, these attempts have had very limited success. Apart from obvious problems such as defining a suitable time origin t, the difficulties with this approach arise from the fact that the secondary compression is very sensitive to the stress history of the peat. Thus, even for an apparently uniform, normally consolidated deposit, variable secondary compression may be expected due to different loadings on the peat which will always occur during construction. Any variation in secondary compression due to differing stress history from one point to another in the field will be further increased by variations in the nature and depth of the peat deposit with position. These factors at least partially explain the observed unevenness of roads over peat which has not been uniformly preloaded.

Additional problems arise in attempting to make predictions of secondary compression based on laboratory tests because overstressing of the peat during sampling can significantly reduce the secondary compression measured in the laboratory, thereby giving a value of C_s which is far too low.

It can be concluded that, in general, it is not possible to obtain reliable predictions of secondary compression based on equations such as Equation 3.5. Nevertheless, secondary compression of normally consolidated peat deposits may be quite large. As noted in Section 2.5, preloading of deposits can significantly reduce the long-term settlement, and the settlement due to the preload can reasonably be estimated using the approach described in Section 3.4. In situations where it is not feasible to preload the deposit, allowance must be made for the fact that large secondary settlements are likely to occur, with or without the use of a geotextile. The magnitude of these settlements is probably best estimated on the basis of local empirical experience.

Geotextiles may be expected to be of some benefit in reducing secondary settlements by reducing overstressing of the peat during construction and, secondly, because they may permit the use of a preload in situations where this might not otherwise have been possible.

3.6/ Pore Pressure Parameters

The excess pore pressures developed within the peat deposit will significantly affect the stability and shear deformations of the peat. The magnitude of the excess pore pressures will be very dependent on the load level, the drainage boundary conditions and the rate of construction. Clearly, accurate prediction of pore pressures for practical applications is impossible. It is possible, however, to adopt an observational approach whereby an embankment is designed on the basis that the excess pore pressures at any time during construction should not exceed a specified design value. Pore pressures can be monitored at critical sections and their magnitude can be controlled by controlling the rate of construction. For design purposes, the maximum excess pore pressure, Δu_{max}, in the deposit can be given by:

$$\Delta u = \bar{B}_{max} \cdot \Delta\sigma_1 \qquad (3.6)$$

where: $\Delta\sigma_1$ = the increase in total stress; and
$\bar{B}_{max}$ = an empirical pore pressure parameter.

As noted in Section 2.4, values of $\bar{B}$ deduced from field case histories typically lie in the range from 0.1 to 0.35, although some higher values have been observed. Considering Skempton's pore pressure equation:

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \qquad (3.5)$$

it is apparent that for conventional CIU triaxial tests where $\Delta\sigma_3$ is zero, $\bar{B}$ as defined by Equation 3.4 is related to A and B by:

$$\bar{B} = B \cdot A \qquad (3.6)$$

CIU tests performed on Bloomington peat (see Appendix E) gave values of B in the range 0.8 to 0.9 and values of A ranging from 0.2 to 0.72 for samples consolidated to 10 kPa or less. The A factor does vary with peat classification but is most sensitive to consolidation pressure. At low confining pressures (e.g., 2.5 kPa) $\bar{B}$ appears to be less than 0.34. Hollingshead and Raymond (1972) have reported A values ranging from 0.36 (σ' = 7 kPa) to 0.77 (σ' = 32 kPa). These results are also consistent with a value of $\bar{B}$ less than 0.34 at typical *in situ* stress levels.

3.7/ Shear Strength of Peat

At low stress levels, the deformation of embankments on peat will be predominantly due to consolidation compression and will largely depend on the deformation parameters described in Section 3.4. At these stress levels, the lateral deformations will be small, as reflected by the low value of Poisson's ratio observed for peat. However, at higher stress levels, significant shear deformation with associated lateral movements and mudwaves may be observed (e.g., see Landva and La Rochelle, 1983). These shear deformations are the result of local plastic failure within the peat and primarily depend on the shear strength parameters for the peat.

There has been considerable discussion in the literature regarding the selection of appropriate shear strength parameters for peat (e.g., Brawner and Milligan, 1967; MacFarlane, 1969; Landva, 1980 and others). Several investigators have performed CIU and CID triaxial tests on peat and have typically reported that peat is an essentially frictional material with a friction angle, ϕ', between 45° and 60° (e.g., Adams, 1961, 1965; Edil, 1981). As part of this current investigation, a series of CIU and CID triaxial tests was performed on Bloomington peat as described in Appendix E. The peat tested has classification ranging from $H_{3\text{-}5}\ B_3\ F_{1\text{-}2}\ R_{0\text{-}1}$ to $H_{7\text{-}8}\ B_2\ F_{1\text{-}2}\ R_0$.

Clearly defined failure of the peat was not obtained in CID tests. The tests were terminated at a strain of 65%. An envelope to the Mohr circles at this stress level indicates that the material is purely frictional with a friction angle in excess of 54°. It should be noted that these very large strains preclude the formation of a conventional failure plane in the sample; consequently, the apparent shear strength reported here, and by others, based on triaxial tests should be viewed with considerable caution.

The CIU triaxial tests also involved large strains prior to failure. These test results also suggest that the two peats tested, which had classifications $H_{3\text{-}5}\ B_3\ F_{1\text{-}2}\ R_{0\text{-}1}$ and $H_{7\text{-}8}\ B_2\ F_{1\text{-}2}\ R_0$ were frictional materials with a friction angle ϕ' of approximately 45°.

The friction angles deduced from triaxial tests on peat seem unreasonably high and, indeed, the use of these parameters in an analysis of the Bloomington embankment (Appendix D) gives a poor prediction of the embankment behaviour.

To provide additional information concerning the shear strength of the Bloomington peat, a series of drained direct shear tests and a series of simple shear tests were conducted as described in Appendix E. The direct shear tests did not give a clearly defined failure at strains less than 13%. Equipment limitations prevented the tests being carried to larger strains. The shear strength envelope deduced by assuming that failure corresponded to the point where failure occurred over most of the failure plane (as evidenced by a change in the stress-strain behaviour) but before failure of a few coarse fibres (which continued to behave "elastically" until the end of the tests), gave a cohesion intercept c = 3 kPa and a friction angle ϕ = 27° (see Figure 3.10) for this surface peat which had a classification $H_{4\text{-}6}\ B_3\ F_2\ R_{1\text{-}2}$.

Two types of simple shear tests were conducted. In the consolidated constant vertical stress tests, the vertical stress remains constant during shearing although the volume may change. The consolidated constant volume (CCV) tests are meant to simulate undrained conditions and in this test the vertical stress is varied such that the constant volume requirement is enforced. The change in vertical stress may be related to the pore pressures which would be developed in a true undrained test. Both tests are run sufficiently slowly to prevent the development of excess pore pressures.

Failure was clearly defined in all cases. For example, Figure 3.10 shows the results obtained from constant vertical stress tests conducted on the surface peat which had a moisture content of 527% and a classification $H_{4\text{-}6}\ B_3\ F_2\ R_{1\text{-}2}$. The failure envelope deduced from these tests is shown in Figure 3.9. The stress paths from the CCV tests on this peat are also shown. The two types of simple shear tests provide very consistent results and the strength envelope has a cohesion intercept c = 1.1 kPa and a friction angle ϕ = 26° These results may be compared with those obtained from the direct shear tests which are also shown.

A second series of tests were conducted on a far more humified peat from a depth of 5 m. This peat had an average moisture content of 160% (as compared to 527% for the peat discussed above) and a classification $H_{7-8}\ B_2\ F_{1-2}\ R_0$. The strength envelope deduced from the CCV simple shear tests had c' = 1.4 kPa and ϕ' = 26°. The parameters may be compared with c' = 0 and ϕ' = 44° obtained from CIU tests on this peat.

Of particular note is the fact that peats with very different moisture contents, degree of humification, and coarse fibre content gave such similar shear strength parameters in all tests.

Figure 3.9/ Mohr-Coulomb Envelope for Peat

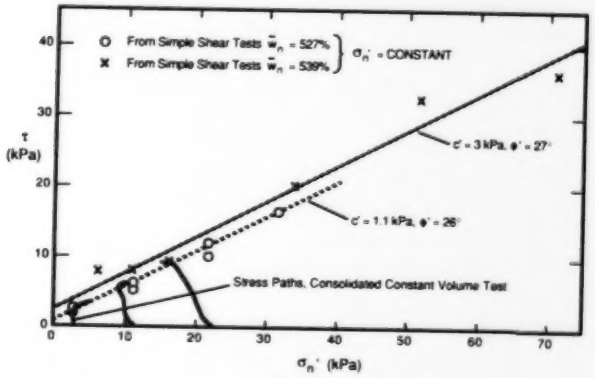
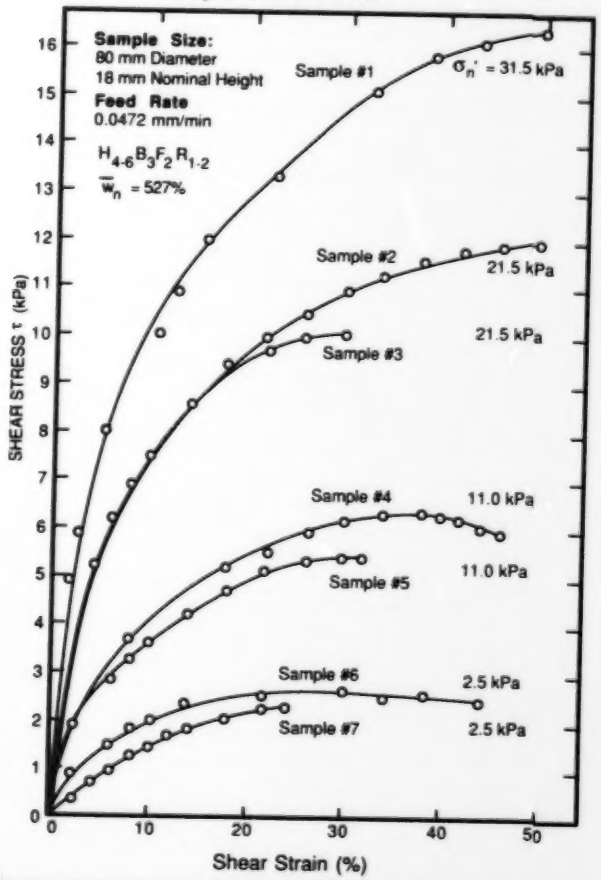

From Simple Shear Tests $\bar{w}_n$ = 527%
From Simple Shear Tests $\bar{w}_n$ = 539%
σ_n' = CONSTANT
c' = 3 kPa, ϕ' = 27°
c' = 1.1 kPa, ϕ' = 26°
Stress Paths, Consolidated Constant Volume Test
τ (kPa)
σ_n' (kPa)

Figure 3.10/ Stress-Strain Curve from Simple Shear Tests



Based on an evaluation of all the available data relating to the strength of Bloomington peat (including tension tests discussed in the next section), the parameters c' = 1.8 kPa and ϕ' = 27° were selected as being representative. In an analysis of the Bloomington embankment (see Appendix D) these parameters gave excellent agreement between observed and predicted behaviour.

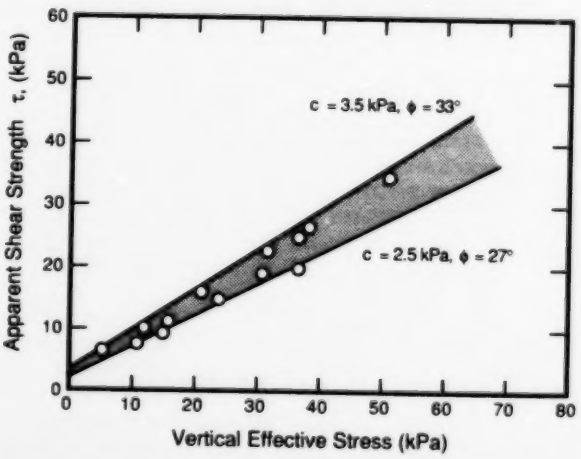
Landva (1980; Landva and La Rochelle, 1983) performed an extensive study of the shear strength of a sphagnum peat from Escuminac. The results from ring shear tests on this peat, which typically had moisture contents in the range from 1000% to 1700% and classification $H_{3-4}\ B_{4-5}\ F_2\ R_2$ are shown in Figure 3.11. These data are bounded by c' = 2.5 kPa, ϕ' = 27° and c' = 3.5 kPa, ϕ' = 33°. A test conducted on a highly decomposed (H_8) peat showed essentially the same behaviour and strength as the H_{3-4} peat. These parameters were all obtained using the ring shear apparatus; however, a simple shear CCV test also conducted on the peat by Landva gave a similar shear strength.

Landva also conducted tests on:
- remoulded peat,
- undisturbed peat sheared at right angles to the *in situ* bedding plane, and
- undisturbed peat with a precut shear plane.

In all cases the shear strength was within the same range (see Figure 3.11) as normally consolidated peat sheared for the first time. This suggests that disturbance or remoulding of peat samples is not critical and that it is the stress and deformation history of the sample that is important (Landva and La Rochelle, 1983).

It is worth noting that Bloomington and Escuminac peats are very different in terms of their basic composition, moisture content and ash content but that the shear strength parameters are very similar. One cannot conclude that all fibrous peats will have shear strengths in the range bracketed by the Bloomington and Escuminac peats; however, an examination of the literature did not reveal any cases where fibrous peat had strength parameters less than those at Bloomington. Furthermore, the shear deformations at Bloomington and at test fill TR8 at Escuminac are among the greatest reported in the literature. Consequently, it is tentatively

Figure 3.11/ Strength Envelope Determined from Torsion Ring Shear Tests on Escuminac Peat by Landva



suggested that the shear strength parameters for Bloomington peat represent the best available lower bound to the strength of fibrous peats. Clearly, it would be highly desirable to test other peats to confirm the validity of this tentative suggestion.

3.8/ Field Vane Tests in Fibrous Peat

The field vane has gained wide acceptance as a tool for estimating the undrained shear strength of soft cohesive soils. Provided that allowance is made for anisotropy and rate effects, there is considerable evidence to suggest that shear strength based on the field vane test results can be used to obtain a good indication of the stability of embankments on soft clay. This approach may also be suitable for determining the stability of embankments on highly amorphous organic soils with a high ash content; however, there is considerable evidence to suggest that this approach is not suitable for embankments on fibrous peat deposits.

The measured field vane strength of peat is highly dependent on the size of the vane used in the test (MacFarlane, 1969; Landva, 1980). Based on an extensive evaluation of the field vane test, Landva (1980) concluded that:

> "The vane shear strength measured in both virgin (unloaded) and loaded moss peat is for a large part an artificial strength brought about by angular compression and by radial expansion.
>
> This mode of deformation and failure leads to an abnormally high local fibre strength of the same nature as that in a fibre rope."

and

> "Vane shear testing of fibrous peats with standard size vanes results in the measurement of an artificial fibre strength only, and no distinct failure surface is formed. Such testing is therefore of little engineering use."

Ripley and Leonoff (1961) reported a failure of an embankment on peat which had been designed based on the results of the field vane test. From a post failure analysis, they concluded that the shear strength mobilized in the peat was significantly less (by almost a factor of three) than the vane shear strength. For future design, they suggested the use of the remoulded vane strength since, in this case, this gave the appropriate average reduction in shear strength required to explain the failure. Although this procedure would work for this specific case, its generality must be questioned, particularly in the light of Landva's work.

Raymond (1969) has also reported a case where failure occurred for an embankment which, on the basis of field vane data, had a factor of safety in excess of 1.5.

To provide a little more data regarding the applicability of the field vane test, a number of vane tests were conducted in conjunction with the 152 mm diameter piston sampling at Bloomington Road and the results are given in Appendix E. A comparison of the field vane results with those obtained from laboratory CIU triaxial tests, unconfined compression tests, simple tension tests, and consolidated constant volume simple shear tests showed that in all cases the field vane strengths were well in excess of those measured in all laboratory tests.

The findings from this study provide further evidence to support the contention that the field vane may greatly overestimate the *in situ* shear strength of peat.

It should be noted that the problems associated with the vane test in peat only serve to emphasize that peat does not exhibit conventional undrained behaviour and that, really, the undrained strength of peat is irrelevant, even if it could be determined. As mentioned in other sections of this report, peat will experience significant pore pressure dissipation during construction and will also undergo large short-term settlements. For these reasons alone, a conventional $\phi_u = 0$ stability analysis is quite inappropriate for fibrous peats.

3.9/ Tensile Strength of Peat

The non-zero cohesion intercept noted in the early discussion of the shear strength of peat appears to result from fibre entanglement and the tensile strength of the fibres. Consequently, it might be expected that the tensile strength of the peat would provide some crude measure of the intercept c. To test this hypothesis, a number of simple tension tests were performed on Bloomington peat as described in Appendix E. These tests, which were conducted on different classifications of peat, did not show any strong anisotropy and although there was some scatter of the tensile strength, the average tensile strength of 1.5 kPa was reasonably consistent with the shear strength envelope deduced from simple shear tests. However, the use of this test for estimating the cohesion intercept may result in considerable error unless special care is taken and numerous tests are conducted so as to provide a reasonable average tensile strength of the deposit.

3.10/ Strength and Deformation Parameters for Clays

For typical rates of construction, the behaviour of soft clay beneath the embankment may be regarded as being undrained. End of construction stability will usually govern the design of these embankments and thus the effect of any geotextile reinforcement will depend on both the undrained shear strength and the undrained modulus. (The modulus is of lesser importance but it will affect the strains developed within the fabric.) Both sets of parameters can be established using techniques such as the field vane and laboratory triaxial tests. However, it should be emphasized that disturbance may significantly reduce both the measured shear strength and modulus. For this reason, samples obtained with 50 mm diameter Shelby tubes should be suspect. Also, it is the author's experience that unconfined compression tests tend to exaggerate the effects of any sample disturbance. While disturbance generally lowers the shear strength and the results are conservative, it should not be ignored since it may significantly affect the design and, consequently, the cost of construction.

The ratio E_u/c_u will vary from clay to clay and may be particularly sensitive to disturbance. The ratio may vary from 50 to 500 for very soft organic or inorganic clays. In the absence of better information, a value of 125 is suggested for preliminary design purposes when only the undrained shear strength has been measured.

4/ Finite Element Analysis of Embankments on Soft/Weak Foundations

4.1/ General

A large number of finite element analyses have been conducted as part of this study. This section provides a brief description of the method of analysis followed by a summary of the results of a parametric study which was conducted to provide an indication of:

- the effect of geotextile modulus;
- the effect of peat thickness;
- the effect of the presence of a soft layer of non-fibrous cohesive soil beneath the peat;
- the effect of lightweight fill used alone and in conjunction with a geotextile; and
- the effect of berms used alone and in conjunction with a geotextile;

upon the performance of low granular embankments constructed on a fibrous peat. Finally, a series of analyses conducted to provide a basis for developing design guidelines for embankments constructed on soft cohesive soils is summarized.

To the extent possible, the method of analysis was validated by examining documented case histories involving the construction of geotextile reinforced embankments on soft cohesive soils and on fibrous peat. The analysis of these cases is described in Appendices A, B and D, and the conclusions are summarized in Section 2.9.

4.2/ Method of Analysis

Several investigators (e.g., Andrawes *et al.*, 1980, 1982; Bell *et al.*, 1977; Ohta *et al.*, 1980; Rowe, 1982, 1984; Boutrup and Holtz, 1983) have used finite element techniques for the analysis of geotextile-soil systems. Despite some similarities, these approaches differ considerably with regard to the modelling of nonlinearity, plastic failure, large deformations, the geotextile, and the geotextile-soil interface conditions. While each of these formulations may be reasonable under certain conditions, they are not all suitable for the analysis of reinforced embankments on soft or compressible foundations where there may be significant deformation or plastic failure within the soil.

Even though an embankment may be stable, there will often be significant local shear failure within the weak underlying foundation material. Provided that these zones where the shear strength has been reached — referred to as "plastic zones" — are contained or surrounded by soil which is not plastic (i.e., where the shear stress is less than the shear strength), collapse will not occur. However, the deformation of the embankment may be significantly affected by the development of these contained plastic regions. Thus, both deformation and shear strength parameters are required for an elasto-plastic finite element deformation analysis. The collapse height of the embankment is simply the height at which uncontained plastic flow occurs and the shear strain becomes indeterminant. It should be emphasized that an embankment may be considered to have failed due to excessive settlement (which is a serviceability assessment) even though collapse, in the sense of uncontained plastic flow, has not occurred.

For embankments on compressible soils, it is important to take account of the effects of large deformations. These effects include a decrease in compressibility as the void ratio decreases and a change in pore pressure head as points in the soil change their position relative to the water table. While considering these factors, it is also essential to ensure that the total stress distribution always satisfies conditions of equilibrium. This latter requirement is intuitively obvious, but can be easily violated by finite element computer codes unless particular care is taken.

The results presented in this report were obtained using the author's plane strain, nonlinear, Large strain, Elasto-Plastic Soil Structure Interaction Analysis program (LEPSSIA) which is based on the general soil structure interaction technique proposed by Rowe *et al.* (1978) but with appropriate modifications for large deformations. These large deformation modifications were based on the complete theory for finite consolidation of an elasto-plastic soil proposed by Carter *et al.* (1979) but involved the following two simplifications. Firstly, large strain rotational effects were neglected (i.e., terms involving the skew symmetric spin tensor were omitted). Secondly, instead of analysing the full development and dissipation of excess pore pressures using Biot theory, separate analyses were performed for the limiting cases of immediately after construction and a long time after construction.

Soft organic/inorganic soils were assumed to exhibit undrained behaviour during the construction period. They were analysed using the undrained shear strength c_u, undrained modulus E_u, a friction angle ϕ of zero, and Poisson's ratio υ of 0.48.

Since peats experience significant dissipation of pore pressures during construction, effective stress analyses were performed for both the end of construction and long time after construction cases. The excess pore pressures immediately after construction were calculated from the equation:

$$\Delta u = \bar{B}\Delta\sigma_1 \qquad (4.1)$$

where: Δu is the excess pore pressure at a point;
$\Delta\sigma_1$ is the increase in total stress at that point;
$\bar{B}$ is an empirical pore pressure parameter.

In the analysis for a long time after construction, it was assumed that all excess pore pressures had dissipated.

The strain increments within the soil were related to the increments in effective stress by means of a nonlinear elasto-plastic stress-strain law. The "Young's modulus" of the soil varied as a function of stress level as described in Chapter 3. The soil had a Mohr-Coulomb failure criterion and plastic

strain rates were related by a flow rule of the form proposed by Davis (1968). Details regarding the elasto-plastic formulation are given by Rowe (1984).

The geotextile was treated as a structural membrane with a nonlinear axial stiffness and negligible flexural rigidity. Provision was made for slip between the geotextile and the soil above or below the geotextile. At each point on the interface, the displacement of the fill and geotextile were compatible until the shear stress reached the shear strength defined by a Mohr-Coulomb criterion at the interface. On reaching the shear strength, slip (i.e., differential tangential displacement between the soil and geotextile) occurred at this location. For each geotextile node, there was a soil node above and a soil node below the geotextile. Thus, slip could occur independently either above or below the inclusion. As part of the large deformation analysis, the co-ordinates of both the soil and the geotextile were updated during the analysis, thereby allowing the development of "membrane forces" due to deformation.

The analyses performed using LEPSSIA and EPSSIA (EPSSIA is the small strain version of the program used for the analyses described in Appendices A and B) involved the simulation of construction including (when appropriate) placing of the working mat, placing of geotextile, the construction of berms, and the construction of the main embankment. The analysis simulated the placing of fill in lifts. The number of lifts required depended upon the settlement of the foundation and the desired height of the embankment. As an example, the analyses performed for Bloomington Road embankment involved nine lifts.

Rowe and Davis (1977) have shown that special attention must be given to numerical procedure if reasonable predictions of collapse loads are to be obtained from finite element analyses. In particular, it is considered essential to adopt a large number of load increments so as to ensure satisfactory convergence of the solution. In large strain analyses, it is insufficient to ensure incremental equilibrium. Rather, it is essential to ensure total equilibrium is satisfied at the end of each load step. In these analyses each load step was split into 20 load increments, thus in the case of the Bloomington embankment a total of 180 load steps were used.

Rounding error may cause problems with nonlinear finite element analyses if 32 or 36 bit wordlength computations are performed. The analyses undertaken as part of this study were all performed on a CYBER 170/835 computer using a 60 bit wordlength.

4.3/ Predicted Behaviour of Low Embankments on a Fibrous Peat Deposit

Analyses were performed for low granular embankments (i.e., height above original ground surface of 2.5 m or less) with a crest width of 13.4 m and 2:1 side slopes resting on peat deposits with depths of 3, 5 and 8 m. The peat parameters adopted (see Table 4.1) were selected on the basis of the literature review and testing described in Chapters 2 and 3. It would be considered unusual for a fibrous peat to have a combination of parameters more critical than those described in Table 4.1, although this possibility cannot be excluded. (Clearly, if there is any doubt for a particular peat, this doubt can be resolved by performing appropriate tests as described in Chapter 3.) Similarly, the properties of the granular fill given in Table 4.2 are considered to be reasonable lower bounds for many granular fills.

The finite element mesh used in these analyses involved between 1472 and 1836 constant strain triangular elements, depending on the depth of the deposit. The lateral boundaries were taken to be smooth and were located between 73 m and 78 m from the centreline of the embankment. The base was assumed to be rough rigid although the shear stresses at the base were not permitted to exceed the shear strength of the soil.

The numerical construction sequence simulated, as nearly as practicable, the construction sequence expected in the field. In particular, it was assumed that berms would be constructed prior to construction of the main embankment as recommended by Raymond (1969). The placement of fill was simulated by placing up to nine layers of fill. Up to 200 load steps were used.

For the end of construction analyses, excess pore pressures were developed as defined by Equation 4.1, but where $\bar{B}$ was assumed to vary with depth and was given by:

$$\bar{B} = (u/u_{max}) \cdot \bar{B}_{max} \qquad (4.2)$$

the variation in (u/u_{max}) is given by the limiting curve in Figure 2.1. Values of $\bar{B}_{max}$ of 0.0, 0.34 and 0.6 were considered. A value of $\bar{B}_{max}$ of zero implies that the embankment was constructed so slowly that no excess pore pressures developed. As indicated in Chapters 2 and 3, $\bar{B}_{max}$ equal to 0.34 is considered the most realistic value. In all cases the water table was assumed to be at original ground level and one-way drainage of excess pore pressures is assumed (i.e., $\bar{B}_{max}$ is at the bottom of the peat).

In the following discussion, the height of the embankment is the height above original ground level. The thickness of the fill is the sum of the embankment height and the settlement.

Table 4.1/ Peat Parameters

c'	1.8 kPa
ϕ'	27°
ψ	0°
υ'	0.15
K_o'	0.176
G_s	1.5
e_o	9
γ	10.3 kN/m^3
E' $(0 \le \sigma_v' \le 20$ kPa)	85 kPa
E' $(20 < \sigma_v' \le 40$ kPa)	110 kPa
E' $(40 \le \sigma_v' \le 60$ kPa)	140 kPa
E' $(\sigma_v' > 60$ kPa)	225 kPa

Table 4.2/ Granular Fill Parameters

c'	0
ϕ'	32°
ψ	0°
υ'	0.35
γ	21 kN/m^3
(E'/p_a)	$100\sqrt{(s_3/p_a)}$
p_a	100 kPa

4.3.1/ Peat Underlain by a Firm Base Stratum

Empirical evidence (e.g., see Chapter 2) would suggest that low embankments on peat can be constructed without collapse provided that:

- the embankment is constructed sufficiently slowly; and
- the peat is underlain by a firm stratum.

The rate of construction is important in that it controls the excess pore pressures developed during construction. To illustrate the effect of these excess pore pressures, analyses were performed from $\bar{B}_{max}$ equal to 0.0, 0.34 and 0.6, and some typical results obtained for 5 m of peat are shown in Figure 4.1.

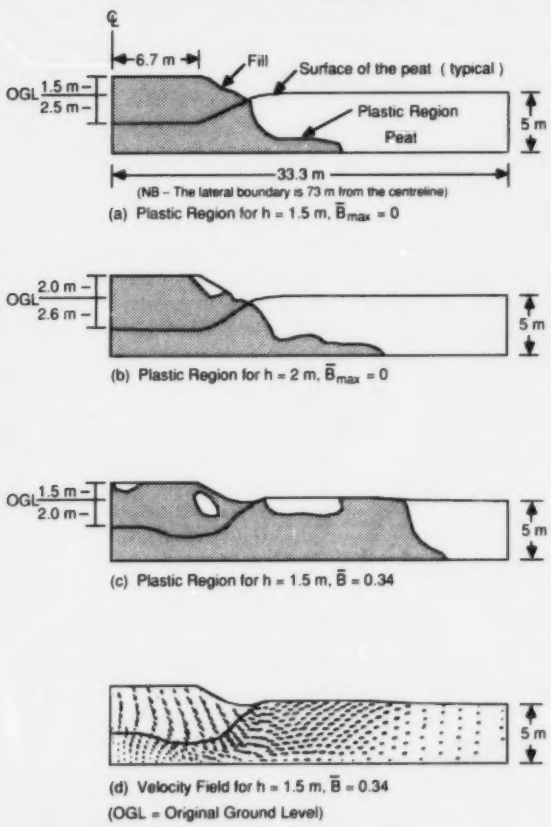
In finite element analyses, collapse of an embankment is indicated when:

- a small increment in load gives a very large increase in the deformation of the embankment (i.e., adding more fill causes a decrease in the embankment height above the original ground level over the portion of the embankment which has failed); and

- the mode of deformation and the plastic region indicate the presence of a collapse mechanism.

The shaded regions in Figure 4.1 show where the soil has reached its shear strength and is in a state of plastic failure. Figure 4.1a shows the plastic region for a granular embankment constructed to a height of 1.5 m above the original ground level, assuming $\bar{B}_{max}$ is zero. This corresponds to drained loading and the end of construction

Figure 4.1/ Embankment Constructed on 5 m of Peat — Firm Base



settlements are the final settlements (neglecting secondary compression). It is apparent that the embankment can be safely constructed to this level although the settlement of 2.5 m is quite large. Figure 4.1b shows essentially the same case but where the fill height is increased to 2 m above the original ground level. This additional fill results in a slight increase in the extent of the plastic region and a little more settlement — but, again, the embankment is quite stable.

Figures 4.1c and 4.1d show the result of an attempt to construct the embankment to a height of 1.5 m assuming $\bar{B}_{max}$ is 0.34. Because this is a partially drained analysis, the settlement at the centreline is less than that for the same height of fill (above original ground level) obtained in the drained analysis (Fig. 4.1a). However, an inspection of the plastic region (Fig. 4.1c) and the velocity field (Fig. 4.1d) shows incipient collapse. The arrows in the velocity field show the direction and relative magnitude of the soil displacements caused by a small increment in load (i.e., a little more fill). Superimposing Figure 4.1d on Figure 4.1c would indicate a collapse mechanism involving an essentially rotational failure which starts approximately 3.5 m from the centreline on the embankment. Of particular note is the fact that the vertical deformations beneath the shoulder of the embankment are greater than at the centreline. This arises because the relatively shallow depth of the deposit forces the large shear deformations to occur near the edge of the embankment. For a deeper deposit, a larger portion of the embankment would be involved in any collapse. It should also be noted that once a collapse mechanism develops, the displacements within the soil mass undergoing plastic deformation are not determinant and the results such as shown in Figure 4.1c only represent a "snapshot" of the deformed shape as collapse takes place (i.e., the displacements become larger and larger without the solution converging).

In summary, it was found that for 5 m of peat, embankments could be:

- constructed to 2.5 m without collapse for $\bar{B}_{max}$ equal to zero;
- constructed to 1.0 m without collapse for $\bar{B}_{max}$ equal to 0.34 (collapse occurred at a height of 1.4 m); and
- could not be constructed to 1.0 m without collapse for $\bar{B}_{max}$ equal to 0.6.

In general, it will not be practical to construct embankments so slowly that no excess pore pressures develop (i.e., $\bar{B}_{max}$ = 0.0) and serious problems can be anticipated if the embankment is constructed at a rate such that $\bar{B}_{max}$ is greater than 0.34. Consequently, it is recommended that embankments be constructed such that $\bar{B}_{max}$ is less than or equal to 0.34 and all the following results and discussion will be for the case where $\bar{B}_{max}$ is equal to 0.34. (These results will be conservative for $\bar{B}_{max}$ less than 0.34.)

The effect of a single layer of geotextile located at or near the interface between the fill and the peat is shown in Figure 4.2 for a height of fill of 1.5 m at the end of construction assuming $\bar{B}_{max}$ is 0.34.

As previously noted, an unreinforced embankment (Fig. 4.2a) could not be safely constructed to a height of 1.5 m for the assumed conditions. The embankment could be constructed using a geotextile with a modulus greater than 150 kN/m and a geotextile-soil interface friction angle in

Figure 4.2/ Deformed Profile and Plastic Regions at the End of Construction for h = 1.5 m, $\bar{B}$ = 0.34

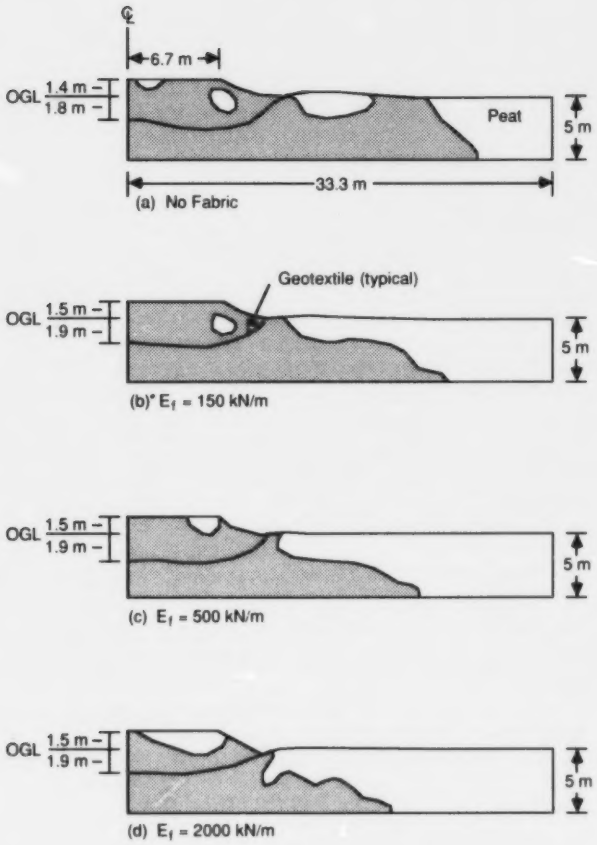
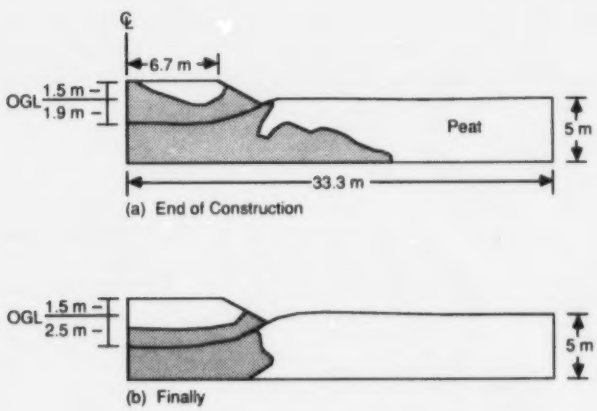


Figure 4.3/ Deformed Profile and Plastic Region for h = 1.5 m, $\bar{B}$ = 0.34, Using a Geotextile with E_f = 2000 kN/m



excess of 20°. For the three cases examined in Figures 4.2b to 4.2d, the centreline settlement was the same; however, increasing the geotextile modulus did improve the performance of the embankment — as may be appreciated by comparing the size of the plastic regions and the deformations of the peat-fill interface beneath the shoulder of the embankment. For example, in Figure 4.2b the maximum vertical deformation occurs near the shoulder of the embankment and although collapse has not yet occurred, the embankment is clearly close to its maximum height using this geotextile. A geotextile with a modulus, E_f, of 2000 kN/m gives a far more satisfactory, dish-shaped settlement profile, as shown in Figure 4.2d.

It was found that the end of construction case represents the most critical situation with regard to stability. This is illustrated in Figure 4.3 which shows the deformed profile and plastic region for a 1.5 m high embankment ($\bar{B}_{max}$ = 0.34, E_f = 2000 kN/m) at the end of construction and finally (i.e., after excess pore pressures have dissipated and the embankment is brought back to grade). Although the final settlements are larger, the plastic region is smaller. For this reason, the following discussion is largely restricted to the behaviour at the end of construction.

Figures 4.4 to 4.6 show the deformed profiles and plastic regions at the end of construction for embankments with heights of 1.5, 2.0 and, where possible, 2.5 m. With a geotextile modulus of 150 kN/m, the embankment could be safely constructed to 1.5 m but the behaviour is quite unsatisfactory at a height of 2 m (see Figs. 4.4b and 4.4c). For E_f of 500 kN/m (Fig. 4.5), the embankment can be safely constructed to 2 m but contiguous plastic failure is imminent at 2.5 m. If conditions were slightly better than those assumed, the embankment probably could be constructed to 2.5 m using a geotextile with E_f of 500 kN/m; however, the possibility of failure cannot be excluded and particular care would be required in monitoring the construction so as to prevent collapse. Figure 4.6 shows that the embankment could be safely constructed to 2.5 m using a geotextile with E_f of 2000 kN/m. Interface slip was not a problem in any of the cases analysed, provided that the interface friction angle exceeded 20°.

The results shown in Figures 4.2 to 4.6 and summarized in Table 4.5a demonstrate that a single layer of geotextile can significantly improve embankment stability and performance. However, geotextiles represent only one design possibility. Alternative designs could involve the use of berms or lightweight fill (i.e., sawdust). To provide a basis for comparison, a number of analyses were also performed assuming the same peat properties and $\bar{B}_{max}$ of 0.34 using berms and lightweight fill.

Raymond (1969) has suggested that when berms are used, they should be constructed prior to the main embankment. Analyses tend to support this recommendation since it was found that an outside-inside construction sequence tended to reduce deformations (see Appendix A). However, consideration should also be given to the effect of berm construction upon excess pore pressures. Ideally, the time between construction of the berms and the main embankment should be sufficient to allow dissipation of excess pore pressure resulting from berm construction. Recognizing that this may not happen in practice, the berm analyses described below were performed assuming $\bar{B}_{max}$ of 0.34 for both the berm and main embankment construction.

Figure 4.4/ Deformed Profile and Plastic Region for $\bar{B}$ = 0.34, Using a Geotextile with E_f = 150 kN/m

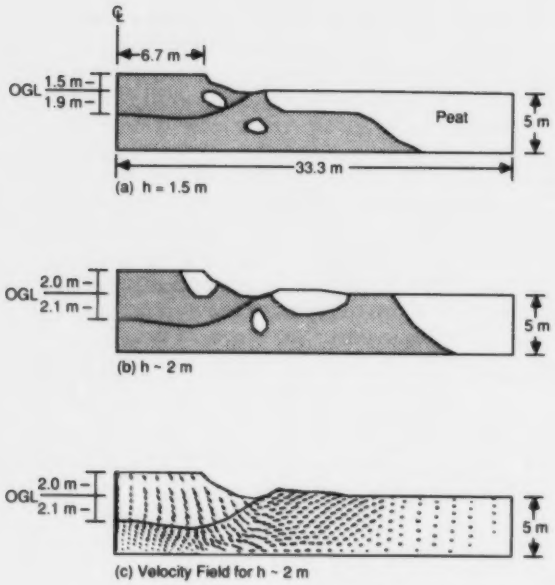


Figure 4.5/ Deformed Profile and Plastic Region for $\bar{B}$ = 0.34, Using a Geotextile with E_f = 500 kN/m

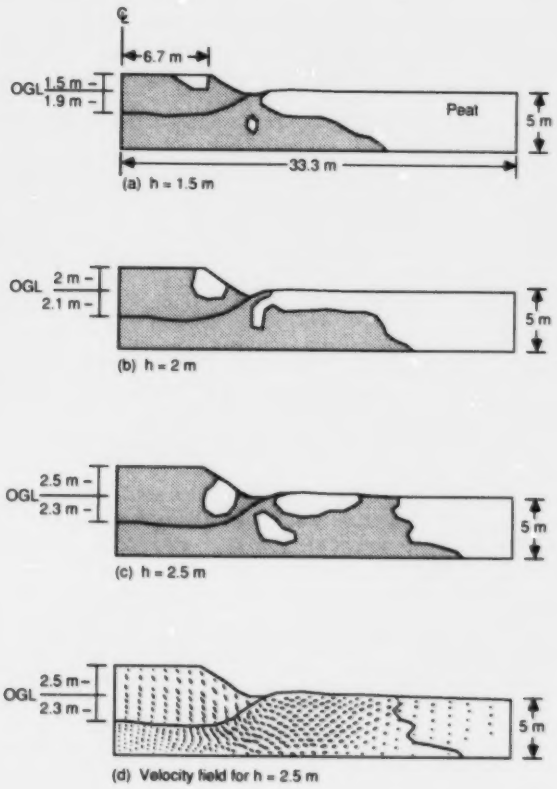


Figure 4.6/ Deformed Profile and Plastic Region for $\bar{B}$ = 0.34, Using a Geotextile with E_f = 2000 kN/m

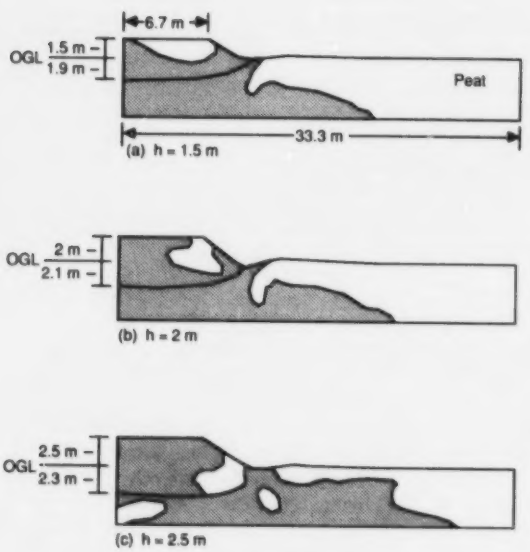


Figure 4.7/ Deformed Profile and Plastic Region, $\bar{B}$ = 0.34, h = 1.5 m

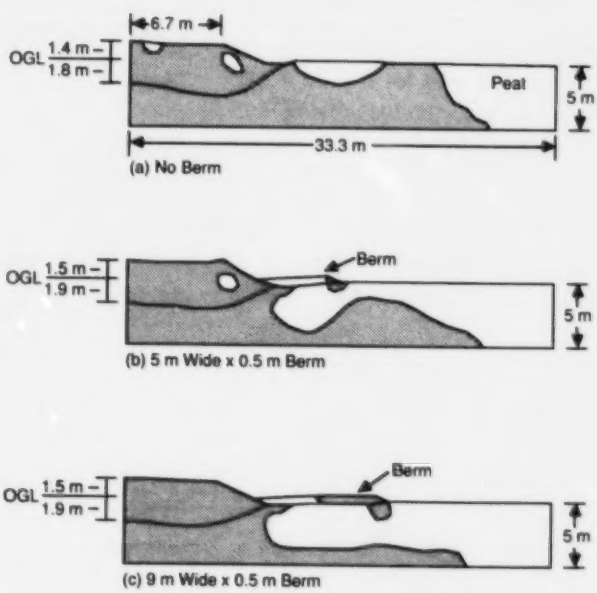


Figure 4.7 shows the effect of 5 m and 9 m wide berms 0.5 m thick on the deformed profile and plastic region for a 1.5 m high embankment. A conventional 1.5 m high embankment could not be constructed under the assumed conditions. A 5 m wide berm would have facilitated construction to a height of 1.5 m but an inspection of the plastic region in Figure 4.7b shows that the embankment is close to collapse. A 9 m wide berm gives a far more satisfactory behaviour which is comparable to that obtained using a geotextile with a modulus of 500 kN/m (see Figure 4.2c). In fact, the embankment could be constructed to 2 m using either a 10 m wide (0.5 m thick) berm or a 500 kN/m geotextile. All other things being equal, the use of the geotextile is the preferred alternative since it will also serve to provide reinforcement of local weak spots (thereby reducing fill requirements and associated construction problems) and possibly improve embankment performance by allowing better compaction of the fill.

Lightweight fill represents a third design alternative. Figure 4.8 shows the deformed profile and plastic region for a 1.5 m high embankment constructed using 1.5 m (initial compacted thickness) of lightweight fill with properties as given in Table 4.3. The results indicate that this embankment could be constructed under the assumed conditions. At the end of construction (Figure 4.8a), the top of the sawdust is just above ground level. After dissipation of excess pore pressures and bringing the embankment back to grade (Figure 4.8b), the top of the sawdust is approximately 0.7 m below the water table. In this case, the use of sawdust reduces final settlement by approximately 25% as compared with an embankment of the same height constructed with either a geotextile or berms.

Given its availability, lightweight fill would appear to be the preferred design alternative; however, some caution is required in adopting this approach. To prevent poor embankment performance and environmental problems due to aerobic deterioration of the sawdust, it is essential that the top of the sawdust be at, or below, the water table at the end of construction and that the surface of the sawdust be below the seasonal minimum height of the water table after dissipation of excess pore pressures and bringing the embankment to grade.

Achieving this requires that the designer be able to make good predictions of the settlements. If the thickness of sawdust is grossly underestimated, too much conventional fill will be required and this may cause collapse. If the thickness of sawdust is overestimated, then the sawdust will be above the water table and either the fill must be removed and the amount of sawdust reduced (which would be inconvenient and expensive) or additional fill must be added to cause the sawdust to settle below the water table (and this could cause collapse).

These problems can be reduced by adopting an observational procedure to the design (Terzaghi and Peck, 1967). With this approach, the thickness of sawdust would be adjusted during construction, depending on ground conditions. However, it is also considered that the use of a geotextile in conjunction with the sawdust will improve the embankment performance and provide backup against collapse if slight overloading is required to ensure that the sawdust is below the water table.

The amount of sawdust which can be used is limited by the settlement which is expected. Both settlement and stability are related to the height of the embankment; however, situations may arise where instability and/or excessive shear deformations occur even though lightweight fill is used. For example, analyses indicate that problems would occur in attempting to construct embankments to 2 m or 2.5 m using only lightweight fill. However, Figure 4.8c does suggest that the embankment could be safely constructed to 2.5 m using 2.1 m of lightweight fill (initial compacted thickness) in conjunction with a geotextile having a modulus of 500 kN/m.

The foregoing discussion has been restricted to embankments constructed on 5 m of peat. Analyses were also performed for the cases of 3 m and 8 m of peat (see Tables 4.5) and these results indicated the same trends as discussed above for the 5 m of peat except that the berms tend to be

Figure 4.8/ Deformed Profile and Plastic Regions, $\bar{B}$ = 0.34

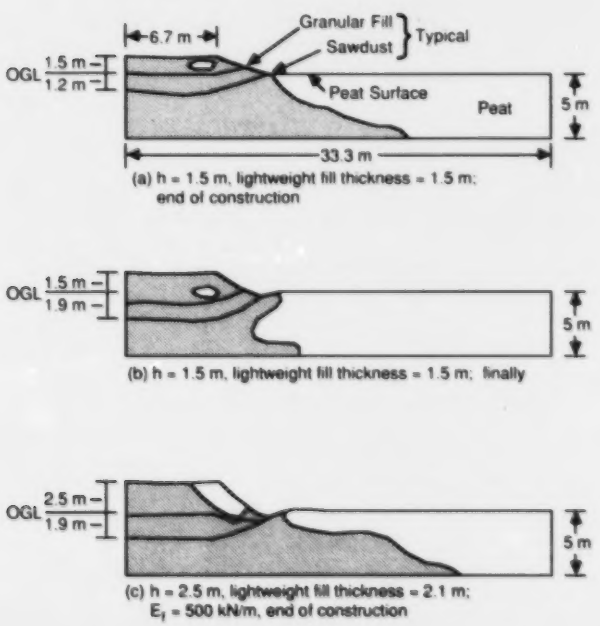


Table 4.3/ Lightweight Fill Parameters

c'	0 kPa
ϕ'	32°
ψ	0°
υ'	0.05
γ	10.2 kN/m³
E'	850 kPa

Table 4.4/ Clay Parameters Used when Underlying Peat

c_u	variable (2.5, 5, 7.5, 10, 15 kPa)
ϕ_u	0°
ψ	0°
υ'	0.48
K_o	0.5
G_s	2.7
e_o	2.5
γ_T	14.6 kN/m³
E_u	1500 kPa

less effective for the deeper deposits. These results have been used in developing the design recommendations given in Chapter 5.

4.3.2/ Peat Underlain by Soft Cohesive Soil

The depositional history of peat deposits is such that they are frequently underlain by soft clayey soils or lake marls. Because the peat has low unit weight, these soft deposits will often have very low shear strength just below the peat, although the strength will generally increase with depth. To illustrate the effect these weak layers can have on stability and performance of low embankments, analyses were performed for a number of cases where the peat was underlain by a clayey soil with parameters as given in Table 4.4. The results of these analyses are summarized in Table 4.5b.

For the purposes of illustrating key points, attention is restricted here to the case of 5 m of peat overlying 3 m of clayey soil with an undrained shear strength of 7.5 kPa. Unless otherwise noted, the pore pressure parameter $\bar{B}_{max}$ in the peat is assumed to be 0.34. The same general trends were observed for other depths of peat and clay, and for other strengths of the underlying clay.

As in the case of peat overlying a firm base, the excess pore pressures developed in the peat can have a significant effect on shear deformations and stability of embankments on peat overlying soft soil. Figures 4.9a and 4.9b show that if the embankment is built sufficiently slowly such that no excess pore pressures develop in the peat (i.e., $\bar{B}_{max} = 0.$), then embankments could be constructed to 1.5 and 2 m under the assumed conditions. However, a comparison of Figures 4.1 and 4.9 shows that even under these ideal conditions, the presence of the clay has significantly increased the shear deformations and the amount of settlement.

Recognizing that excess pore pressures will invariably develop in the peat, an analysis was also performed for $\bar{B}_{max}$ equal to 0.34. As shown in Figures 4.9c and 4.9d, the embankment could not be constructed to a height of 1.5 m. The sloping surface of the crest is a direct result of collapse occurring, as illustrated by the almost circular failure mechanism apparent in Figure 4.9d. Introducing a geotextile improves the embankment stability and Figure 4.10a shows that the embankment could be constructed to 1.5 m if a geotextile with E_f = 2000 kN/m were placed at, or near, the interface between the peat and the fill. However, there is a limit to how much even a very high modulus fabric can achieve, as illustrated by Figures 4.10b and 4.10c which show that a 2000 kN/m geotextile would not permit construction of the embankment to a height of 2 m. It is particularly important to note that the stiff geotextile does significantly modify the failure mechanism (compare Figures 4.9d and 4.10c) by almost eliminating lateral deformations of the embankment and, in this case, collapse is governed by lateral squeeze primarily within the soft clay layer. The performance of this embankment could not be improved by using an even stiffer geotextile and a different design and/or construction procedure would be required to achieve a 2 m grade under the assumed conditions.

Lightweight fill may be used to increase embankment stability; however, the associated difficulties discussed earlier are magnified by the presence of a soft clay layer. Experience has shown (e.g., Raymond, 1969, and others) that the use of lightweight fill alone may not be sufficient to ensure stability and that other stabilizing measures such as berms (or geotextiles) may be necessary. This problem is illustrated in Figures 4.11a and 4.11b which show that an embankment could not be constructed to 2 m using 2 m of lightweight fill. Again, the sloping crest in Figures 4.11a and 4.11b is an indication that it was not possible to construct the embankment to grade. The use of a 500 kN/m modulus

Table 4.5 (a)/ Results from Analyses for Peat Underlain by a Firm Base, $\bar{B}_{max}$ = 0.34

D	h	E_f (kN/m)				
(m)	(m)	0	150	500	1000	2000
3	1.0	Y		Y		
	1.5	Y		Y		
	2.0	N		Y		
	2.5	N		YM		
5	1.0	Y	Y	Y	Y	Y
	1.5	NM	Y	Y	Y	Y
	2.0	N	N	YM	Y	Y
	2.5	N	N	N	YM	Y
8	1.0	Y	Y	Y		Y
	1.5	N	N	Y		Y
	2.0	N	N	YM		Y
	2.5	N	N	N		YM

Notes: Y = Embankment could be constructed to desired height, h.
YM = Could just be constructed to height h — potential problems.
NM = Severe problems could be expected at this height, or fails just prior to acheiving height h.
N = Could not be constructed to height h.
h = Embankment height above original ground level (OGL).

geotextile beneath the 2 m (initial compacted thickness) of sawdust significantly improved the performance of the embankment and permitted construction to a height of 2 m, as shown in Figure 4.11c. This figure also shows that the optimal amount of sawdust was used. After placement of the 2 m of granular fill above the sawdust, it had compressed to 1.8 m and the top of the sawdust at the end of construction was coincident with the water table. Similar analyses showed that the embankment could be constructed to 2.5 m by using 2.2 m of lightweight fill in conjunction with a 2000 kN/m modulus geotextile.

An alternative to the use of lightweight fill would be the use of berms. Analyses showed that for the assumed conditions, an embankment could be constructed to a height of 2 m by first constructing berms 16 m wide by 1 m thick. A smaller berm could be used in conjunction with a geotextile and Figure 4.12 shows that an embankment could be constructed to 2 m by using a berm 5 m wide by 1 m thick in conjunction with a 2000 kN/m geotextile located at, or near, the interface between the fill and peat. An examination of Figure 4.12 also suggests that the use of a wider berm would be highly desirable.

The results presented in this section serve to indicate that situations may occur where the use of even a very stiff geotextile may not be sufficient to ensure stability of low embankments on peat overlying soft clay. All other things being equal, the use of a geotextile in conjunction with lightweight fill appears to be the most satisfactory design option since this minimizes settlements and shear deformations while providing the additional stability required to ensure that the sawdust is submerged. Geotextiles used alone will provide some increase in stability but this may not be enough in many cases. The geotextile may also be useful in reducing the size of berm required to ensure stability.

4.4/ Predicted Behaviour of Low Embankments on a Cohesive Deposit

The previous section was concerned with the behaviour of embankments on fibrous peat which will experience significant pore pressure dissipation during construction and where there will be large "end of construction" settlements. In this section, consideration is given to the behaviour of embankments on very soft organic or inorganic soils characterized by low to negligible fibre content, high ash content

Table 4.5 (b)/ Results from Analyses for Peat Underlain by a Soft Cohesive Soil, $\bar{B}_{max} = 0.34$

Depth of Peat		**3 m**			**5 m**		
Depth of Soft Layer		**2 m**			**3 m**		
Shear Strength of Soft Layer							
c_u	h	E_f (kN/m)			E_f (kN/m)		
(kPa)	(m)	**0**	**500**	**2000**	**0**	**500**	**2000**
15	1.0	Y	Y	Y	Y	Y	Y
	1.5	Y	Y	Y	NM	Y	Y
	2.0	N	Y	Y	N	YM	Y
	2.5	N	YM	Y	N	N	Y
10	1.0	Y	Y	Y	Y	Y	Y
	1.5	Y	Y	Y	NM	Y	Y
	2.0	N	Y	Y	N	N	Y
	2.5	N	YM	Y	N	N	N
7.5	1.0	Y	Y	Y	NM	Y	Y
	1.5	YM	Y	Y	N	NM	Y
	2.0	N	Y	Y	N	N	N
	2.5	N	N	Y	N	N	N
5.0	1.0	Y	Y	Y			
	1.5	N	Y	Y			
	2.0	N	N	Y			
	2.5	N	N	N			
2.5	1.0	Y		Y			N
	1.5	N		Y			N
	2.0	N		N			N
	2.5	N		N			N

Notes:
- Y = Embankment could be constructed to desired height, h.
- YM = Could just be constructed to height h — potential problems.
- NM = Severe problems could be expected at this height, or fails just prior to acheiving height h
- N = Could not be constructed to height h.
- h = Embankment height above original ground level (OGL).

Figure 4.9/ Deformed Profile and Plastic Regions, c_u (Clay) = 7.5 kPa

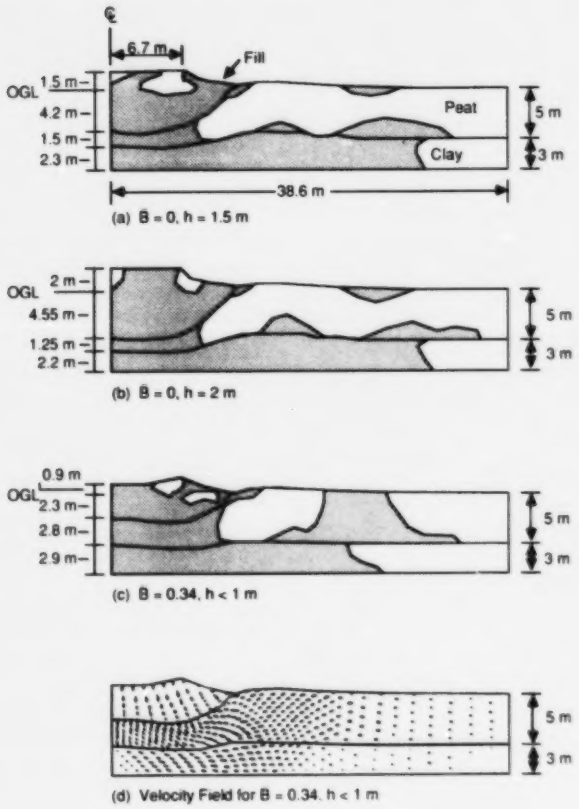


Figure 4.10/ Deformed Profile and Plastic Regions and Velocity Fields, c_u (Clay) = 7.5 kPa, $\bar{B}$ = 0.34

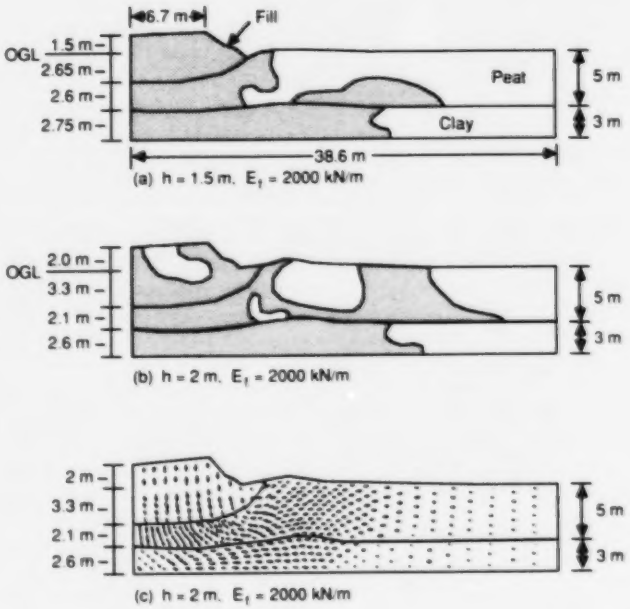


Figure 4.11/ Attempt to Construct Embankment to h = 2 m, c_u (Clay) = 7 kPa, $\bar{B}$ = 0.34, Initial Compacted Thickness of Sawdust = 2 m

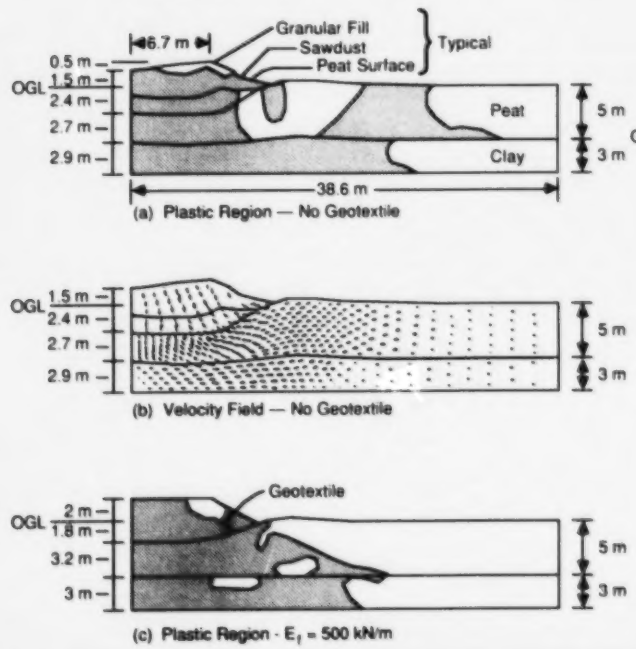


Figure 4.12/ Deformed Profile and Plastic Region, c_u (Clay) = 7.5 kPa, $\bar{B}$ = 0.34, h = 2 m, 5 m wide x 1 m high Berm Geotextile with E_f = 2000 kN/m

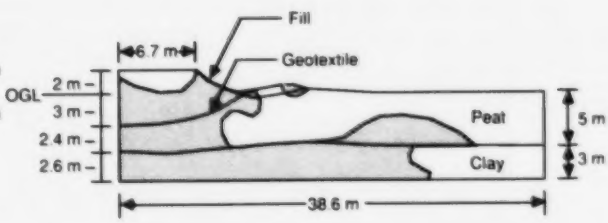


Table 4.6/ Clay Soil Parameters

c_u	variable (3.85, 5, 5.77, 7.5, 10 kPa)
ϕ_u	0°
υ'	0.48
K_o	0.5
G_s	2.7
e_o	variable (3.5, 5.7)
γ_T	variable (13.5, 12.3 kN/m^3)
E_u	variable (500, 1000 kPa)

and which, relative to usual construction rates, have a low permeability and behave essentially as purely cohesive soil under undrained conditions. The behaviour of two geotextile reinforced embankments constructed on this type of soil is described in Appendices A and B.

4.4.1/ Results From Finite Element Analyses

A series of analyses were performed for embankments overlying 3 m and 5 m of soil with properties as given in Table 4.6. The soil was assumed to be homogeneous. For practical situations where the soil is not homogeneous, engineering judgement is required to select representative properties for the deposit. The fill was assumed to be frictional with properties as given in Table 4.2. The finite element mesh used in these analyses involved 1472 constant strain triangular elements. The lateral boundaries were taken to be smooth and were located 73 m from the centreline of the embankment. The base was assumed to be rough rigid but with provision for slip at the base at points where the base-soil shear stress was equal to the shear strength of the soil. Embankment construction was simulated by placing the fill in lifts. Up to five layers were placed involving up to 120 load steps.

In analyses performed with geotextile reinforcement, the reinforcement was assumed to be placed on top of the foundation soil over the entire width of the base of the embankment. The soil-geotextile interface shear strength was assumed to be purely cohesive when in contact with the clayey soil and purely frictional on the side in contact with the fill. In all cases where slip occurred, the cohesive foundation-geotextile shear strength was the controlling factor.

Figure 4.13 shows the plastic region and velocity field at collapse for an unreinforced embankment on a soil with c_u of 7.5 kPa. It can be seen that the collapse mechanism corresponds to a circular type failure surface and in fact the factors of safety determined from the finite element analysis and a conventional simplified Bishop limit equilibrium analysis are, for all practical purposes, identical.

In a rigid-plastic analysis (e.g., limit equilibrium analyses), it is implicitly assumed that there will be no deformation prior to the formation of a collapse mechanism. In an unreinforced embankment, collapse could occur as soon as the soil shear strength is fully mobilized along the potential failure surface. In a reinforced embankment, it is typically assumed (e.g., Jewel, 1982; Fowler, 1982) that once the shear strength is mobilized, load will then be carried by the reinforcement and collapse will not occur until the strength of the reinforcement is exceeded or the geotextile-soil interface strength is reached.

The finite element analysis models the deformations of the soil and the geotextile both before and after the shear strength of the soil has been mobilized. The deformations of an embankment prior to collapse depend on the shear strength and deformation parameters of the foundation, embankment geometry and the depth of the layer. It is obvious that the lower the undrained modulus and the deeper the deposit, the greater will be the deformations of a given embankment. It follows from this that the geotextile strains in a given reinforced embankment will similarly depend on the undrained modulus and depth of the founding layer. Thus, for a given geotextile, the force mobilized by the geotextile will depend on the undrained modulus and depth of the soil and, indeed, its effectiveness will be partly dependent upon the deformations and forces developed in the fabric prior to full mobilization of the soil's shear strength.

The observations of the last paragraph are demonstrated by the finite element results shown in Figure 4.14 for 5 m of soil with a shear strength c_u of 7.5 kPa. The plastic region for an unreinforced embankment at the critical height of 1.95 m is shown in Figure 4.14a. With the use of a 2000 kN/m geotextile, the height at which the shear strength is mobilized along a potential failure surface is increased to 2.26 m. Both of these results were obtained assuming the undrained modulus of the soil to be 1000 kPa. If the analyses are repeated using an undrained modulus of 500 kPa, the critical height for the unreinforced embankment is the same; however, for the embankment with 2000 kN/m modulus geotextile, the height at which the shear strength is fully

Figure 4.13/ Foundation Soil
D = 5 m, c_u = 7.5 kPa, E_u = 100 kPa

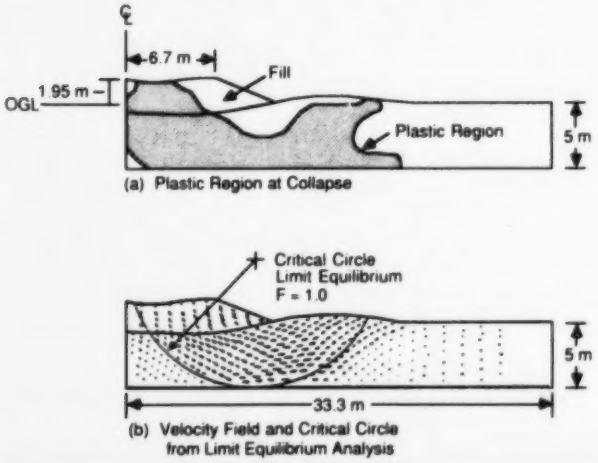
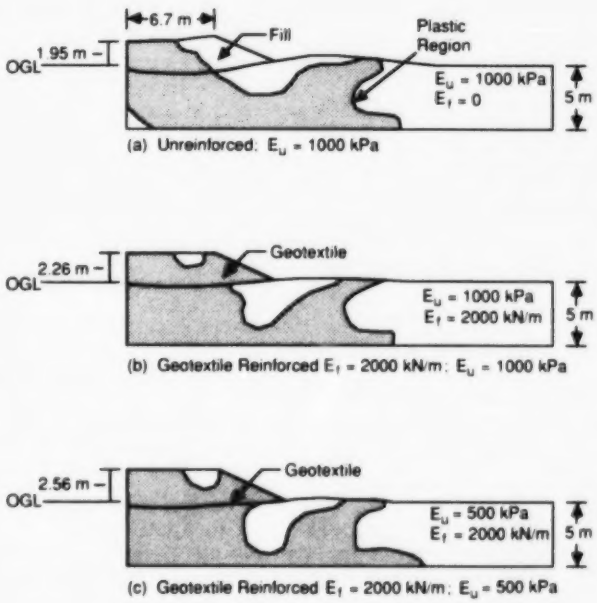


Figure 4.14/ Plastic Regions at Collapse
D = 5 m, c_u = 7.5 kPa



mobilized is increased to 2.56 m. Clearly, the same geotextile is more effective in the second case and this illustrates the fact that any simple design procedure must take account of the deformation that would be expected prior to collapse.

A simple design procedure, based on these finite element results, which takes account of soil-geotextile interaction is described in Chapter 5.

5/ Recommendations for the Use of Geotextiles as Reinforcement for Low Embankments on Soft/Weak Soils

5.1/ General

This section summarizes the findings from this study in terms of recommendations for the use of geotextiles as reinforcement. The background information and data upon which the recommendations are based is given in the previous chapters and additional details are given in the appendices.

The scope of this study has been restricted to the use of a single layer of geotextile (or geogrid) reinforcement in low, two-lane embankments (2.5 m or less above original ground level and 13 m to 14 m wide at the crest) constructed on fibrous peats or very soft cohesive soil. Although many of the recommendations may also be appropriate under other circumstances, the findings should not be extrapolated to situations beyond the scope of this study without independent verification. It should also be emphasized that the recommendations in the report are intended to be used as guidelines by qualified personnel. The information provided in this report should not be used without first securing competent geotechnical advice with respect to its suitability for any general or specific application.

5.2/ Engineering Properties of Geotextiles

On the basis of the examination of published data and the investigation reported in Appendix G, the following recommendations are made with regard to the determination of geotextile stress-strain characteristics to be used in the design of geotextile reinforced embankments.

1) The stress-strain parameters of woven geotextiles and extruded geogrids determined from wide strip uniaxial tensile tests are appropriate for use in the design of reinforced embankments.

2) The properties of non-woven and composite fabrics may be significantly altered by the confining pressure of a soil-geotextile system and parameters determined from uniaxial tensile tests may not be appropriate for use in design. This aspect of geotextile behaviour requires a more detailed investigation. Based on available data from the literature, it would appear that the uniaxial wide strip test may underestimate the strength and modulus of non-woven or composite geotextiles.

3) The engineering properties of polyethylene-based geotextiles (e.g., Tensar SR2) are temperature dependent. However, the ultimate capacity and moduli determined at room temperature should be conservative, provided the geotextile is used at temperatures in the range from 0 to 25°C.

4) In the design of a tensile testing system for geotextiles, the grips should not be allowed to rotate.

5) The gripping system developed for the tests described in this report worked well for all the geotextiles tested.

6) The measured stress-strain characteristics of geotextiles may vary significantly depending on the specimen dimensions and the strain rate. A specimen size with a large aspect ratio and a relatively small gauge length is recommended. It appears that a specimen 500 mm wide with a 100 mm gauge length is appropriate. In general, a strain rate of 2%/min is considered practical for obtaining an indication of design parameters for geotextiles.

7) The modulus and tensile strength are the key design parameters for geotextile reinforcement. The tensile strength and strain at failure should be reported for each test conducted, together with values of modulus corresponding to:
   - the initial tangent modulus
   - the tangent modulus from 2 - 5% strain
   - the tangent modulus from 5 - 10% strain
   - the secant modulus from 0 - 5% strain
   - the secant modulus from 0 - 10% strain

8) The temperature, specimen size and strain rate should be reported for all tests. A suggested test procedure is given in Appendix H.

9) For medium to low strength geotextiles with relatively fine structure, a minimum of three tests is considered necessary to obtain results within 10% precision at 95% probability level. However, for high strength geotextiles with coarse structure, it is suggested that at least six tests should be performed to obtain the same accuracy.

10) The stress-strain properties given in Table 5.1 for the geotextiles examined in this study can be used as an initial indication of the design parameters. It is noted that modulus values are sensitive to strain rate and the results reported here are lower than those reported by the manufacturers (which are based on tests at much higher strain rates).

11) For design purposes, it is suggested that geotextiles (including geogrids) be divided into five classes, viz:

Class	Modulus E_f (kN/m)
A	<150
B	150 to 500
C	500 to 1000
D	1000 to 2000
E	>2000

Unless otherwise specified by the designer, the modulus E_f should be the minimum of the secant modulus determined from 0-5% and 0-10% strain.

12) The grab tension test can be calibrated against the results from wide strip tests for a given geotextile. The grab test can then be used as an index test for quality control. The relationship between the wide strip results and grab test vary from geotextile to geotextile.

5.3/Geotextile-Soil Interface Strength

Geotextile-soil interface shear strength parameters were determined for the cases of:

a) Direct shear — where the soil on one side of the geotextile moves relative to the geotextile and the soil on the other side of the geotextile. This mechanism relates to sliding of the fill along the geotextile. If this interface strength is less than that of the fill alone then the geotextile represents a plane of weakness in the embankment.

o) Pullout — when the geotextile displaces relative to the soil on both sides of the geotextile (i.e., it is pulled out). This is the mechanism which controls the anchorage of the geotextile and hence controls the maximum force that can be developed in the geotextile.

These tests, which were performed to determine the fill-geotextile interface shear strength for a natural silty sandy fill and sawdust (see Appendix F), indicate that:

1) For granular fills and geotextile fabrics similar to that examined in this study, the soil-fabric interface strength is approximately equal to the shear strength of the fill in the loose state, both in direct shear and pullout.

2) For geogrids in loose fill whose grain size is small compared to the openings in the geogrid, the interface shear strength in pullout was significantly less than that of the loose fill alone; special care must be taken to ensure adequate anchorage of the geotextile under these circumstances. It is anticipated that the interface strength may be considerably higher for coarser and/or denser fills and thus it is important that the pullout resistance of the open mesh geogrids be determined for the particular fill and expected fill density for any given project. The interface strength in direct shear was essentially the same as that of the loose fill alone.

3) The sawdust-geotextile interface shear strength was measured for two woven geotextiles. It was found to be considerably less than that of the sawdust alone and was quite sensitive to the roughness of the particular fabric being tested. The sawdust-geotextile interface shear strength should be determined for the proposed geotextile and sawdust for any given project.

Table 5.1a/ Stress-Strain Characteristics of Seven Geotextiles (Strain Rate, 2%/min; temperature 25° ± 1°C)

Geotextile Type	Warp (W) Fill (F)	Maximum Tensile Strength (kN/m)	Strain at Failure (%)	Secant Modulus (kN/m)			Tangent Modulus (kN/m)		
				0-2%ε	0-5%ε	0-10%ε	Initial	2-5%ε*	5-10%ε
Geolon 1250	W	179	24.2	315	458	632	192	672	806
	F	33.8	16.5	117	140	200	100	175	260
Permealiner M1195	W	41.2	35.9	226	161	150	226	119	138
	F	24.8	47.9	131	110	94	131	96	78
Mirafi P600X	W or F	36.1	33.3	181	181	170	181	183	159
Mirafi P500	W	22.1	43.3	75	75	75	75	75	75
	F	23.4	24.15	194	145	126	194	113	108
Tensar SR2	W	60.7	44.1 (17.4)	716	588	474	716	503	359
	F	9.3	>48 (12)	162	148	90	162	139	32
Terrafix 1200R	W	31.8	85.5	31	27	28	31	25	29
	F	23	86	14	13	16	14	13	18
Terrafix 370RS	W	≈12.4 (5.3)	>100 (11.5)	50	50	50	50	50	50
	F	17.5 (4.9)	81 (8.3)	75	73	**	75	71	**

Notes: * 3% for Geolon 1250, number in parentheses indicates strain at first yield.

** Fabric yields before 10%ε.

All values are obtained from tests on specimen size 500 x 100 mm (460 x 216 mm for Tensar SR2).

Table 5.1b/ Stress-Strain Characteristics of Six Additional Geotextiles (Strain Rate, 2%/min; temperature 26° ± 0.5°C)
(In warp direction unless otherwise noted.)

Material		Maximum Tensile Strength (kN/m)	Strain at Failure (%)	Secant Modulus (kN/m)			Tangent Modulus (kN/m)		
				0-2%ε	0-5%ε	0-10%ε	Initial	2-5%ε*	5-10%ε
Stabilenka 220		176 42.9	8.2 16	2250 200	2160 172	Failed 208	2660 200	2100 153	— 244
Terratrak TT II		6.5	15.3	56	56	50	56	56	44
Terratrak TT2411		15.7	42.6	63	63	58	63	63	54
Terratrak TT400		40	36.5	163	163	163	163	163	163
Terratrak TT200		25.8	46	88	88	88	88	88	88
Terratrak TT2415		15.6	43	57	56	55	57	55	54
Tensar SS1	(a) (b)	19.5 13.2	8.2 16.3	410 252	306 171	* 118	536 353	238 1171	* 64
Tensar SS2	(a) (b)	32 20.1	12.9 12.3	597 454	430 294	308 192	710 663	319 186	186 90

Notes: All values are obtained from tests on specimen size 500 x 100 mm, except:
- Tensar SS1, which was 503 x 190 mm (18 x 6 ribs), and
- Tensar SS2, which was 457 x 196 mm (12 x 8 ribs).

(a) = Test direction was along roll width.
(b) = Test direction was along roll length.

* = Specimen failed before 10% strain.

Table 5.1c/ Stress-Strain Characteristics of Three Additional Geotextiles (Strain Rate, 2%/min; temperature 26° ± 0.5°C)

Material		Maximum Tensile Strength (kN/m)	Strain at Failure (%)	Secant Modulus (kN/m)			Tangent Modulus (kN/m)		
				0-2%ε	0-5%ε	0-10%ε	Initial	2-5%ε*	5-10%ε
Mirafi 2300HP	W F	154 160	16.5 13.5	750 1150	750 960	750 1050	750 1150	750 833	750 1140
Mirafi 1500HP	W F	104 95.5	15 10.5	375 1300	550 1100	650 920	300 1500	667 967	750 740
Mirafi 1200HP	W F	64.1 75.5	17 16	440 580	420 550	420 530	440 580	407 530	420 510

Notes: All values are obtained from tests on specimen size 500 x 100 mm.

W = Warp
F = Fill

5.4/ Site Investigation

The design of embankments should be based on a reasonable knowledge of site conditions. In particular, the site investigation should, as a minimum, provide a characterization of the nature and depth of the soil strata which are likely to affect the embankment performance. This should include:

- a description of the ground water conditions;
- a description of the surface cover (e.g., Radforth, 1969) including, where appropriate, an assessment of the nature and general competency of the root mat;
- classification of any peat according to *both* the Radforth classification and the modified Von Post classification. Index tests on the peat should include moisture content and ash content determination. On the basis of a recent review of techniques for determining these two quantities, Landva *et al.* (1983) recommend:
  "...that the water content of organic soils be determined by oven-drying at a temperature not exceeding about 90°C and that the ash content be determined by igniting at a temperature of 440°C for about 5 hours."

These recommendations appear to be well founded.

- depth of any peat layer;
- identification of the presence of any soft clays, marls or silts beneath peat deposits, including an indication of the thickness, strength and extent of these deposits;
- if the embankment is to be constructed on soils which may be expected to exhibit undrained behaviour at the anticipated construction rate (e.g., very soft organic clays), the depth, undrained shear strength and undrained modulus should be determined. Particular care is required in obtaining undisturbed samples of these soils. As a minimum, a 75 mm piston sampler should be used.
- any particularly weak or otherwise potentially troublesome areas should be noted. For example, the soil beneath present or past streams may be particularly soft and devoid of a competent root mat. Also, areas where there is a rapid change in the thickness of the compressible deposit will require special attention to minimize problems due to differential settlement.

5.5/ Testing of Peat

The behaviour of embankments on peat will largely depend on the deformation and shear strength parameters of the peat. The available information (including the work done in this study) would suggest that reasonable values for the compressibility of fibrous peat can be determined from conventional one-dimensional consolidation tests using multi-stage loading (e.g., 3, 6, 12, 24, 48 ...kPa) with load increments being applied every 24 hours. The results from these tests should be presented as a linear plot of void ratio versus applied stress. (The reasons for this are given in Chapter 3.) Provided that representative values are adopted and the embankment is designed to minimize shear deformations, these parameters can be expected to provide reasonable predictions of "consolidation" settlement (i.e., excluding long-term secondary compression).

The secondary compression of peat appears to be largely governed by the stress history of the peat. Since this will vary significantly from point to point in the field, and since sampling will impose a non-representative stress history on peat samples, values of the coefficient of secondary compression determined in the laboratory cannot be expected to provide a good indication of secondary compression which will occur in the field.

Although they may be of some use for determining the modulus and Poisson's ratio of peat, conventional triaxial tests on fibrous peat appear to be of little value in determining shear strength. The shear strength parameters deduced from triaxial tests are not considered appropriate for the design of embankments. More research into the determination of the shear strength of peat is required. However, based on the results of this study, the torsion ring shear and Norwegian simple shear tests appear to give the most appropriate shear strength values. The representative values for Bloomington peat are $c' = 1.8$ kPa and $\phi' = 27°$. Tests by Landva (1980) on a quite different Escuminac peat gave values of $c' = 2.5$ to 3.5 kPa, $\phi' = 27°$ to 33°.

For the Bloomington peat, the tensile strength of the peat determined using 51 mm diameter cylindrical samples appeared to provide a reasonable estimate of the "cohesion" intercept, c', of this peat. More testing is required on other peats to establish the general usefulness of this test.

Fibrous peat generally appears to have a low Poisson's ratio (0.1 - 0.3), low K_o (0.17 - 0.3), and low unit weight (8.6 - 11.8 kN/m). On the basis of his studies, Landva (1980) has concluded that:
"Vane testing of fibrous peats with standard size vanes results in the measurement of an artificial fibre strength only, and no distinct failure surface is formed. Such testing is therefore of little engineering use."

The results of the present study of Bloomington peat support Landva's conclusion that the parameters determined from the vane test on fibrous peat are not relevant to the design of embankments on this material. The vane may still be of some use in assessing the variability of a deposit.

5.6/ Design of Embankments on Soft/Weak Deposits

On the basis of the studies described in Chapter 4, Appendices A to D, and in the interim report for this project (Rowe *et al.*, 1982) it may be generally concluded that:

1) Geotextiles may act as a separator, preventing mixing of fill and foundation material, as reinforcement with respect to local weak spots, and as general reinforcement. The benefits from the separator function are well recognized but difficult to quantify. The benefits achieved from the reinforcement function will depend upon the modulus and strength of the fabric relative to the fill and underlying subsoil.

   The single most important factor to be considered when selecting geotextiles for use as general reinforcement (i.e., to improve overall stability and embankment performance) is the fabric modulus. For a given soil, there is a limiting fabric modulus below which the geotextile has little or no effect as reinforcement. The magnitude of this lower threshold modulus will depend on the compressibility and strength of the fill and foundation material. On the basis of the current research, it is suggested that a minimum acceptable secant modulus over the strain range 0 - 5% is 125 kN/m. This will be sufficient to provide

some reinforcement for embankments on very compressible soils (e.g., peat). There is also an upper limit beyond which any further increase in fabric modulus will not further improve the embankment performance.

2) In the selection of suitable geotextiles, consideration should be given to the nature of the surface upon which it will be placed. Some useful guidelines for the selection of a geotextile with adequate survivability have been suggested by Haliburton and are summarized in Tables 5.2 and 5.3.

3) For typical low granular fills (height of 2.5 m or less; 2:1 side slopes) and geotextiles, where the interface friction angle is greater than 25°, interface slip is unlikely to occur for geotextiles with a modulus of 2000 kN/m or less. The likelihood of slip increases with increasing fabric modulus and under some circumstances will limit the effectiveness of geotextiles for cases where the modulus is in excess of 2000 kN/m.

4) In situations where the geotextile-soil interface is cohesive (e.g., fabric on very soft clay), fabric-soil interface slip may control the stability of a reinforced embankment.

5) The use of a thick working mat decreases the effectiveness of the geotextile. Consequently, the geotextile should be placed as close to the natural ground surface as practical.

6) Geotextiles will not significantly reduce vertical consolidation settlement. For typical embankments on relatively shallow soft deposits (with depth less than the embankment width), an estimate of centreline consolidation settlement may be obtained assuming one-dimensional strain conditions.

7) Local yield within the soil mass will result in settlements greater than predicted assuming one-dimensional strain. The presence of a geotextile may increase embankment stability and reduce local yield with a consequent reduction in settlement. For cases in which the actual factor of safety without geotextile is greater than unity, the inclusion of a geotextile is unlikely to reduce vertical settlement by more than 20%. However, an appropriate geotextile may significantly reduce lateral spreading and increase the factor of safety.

8) The outside-inside construction technique proposed by Haliburton *et al.* (1980) is superior to normal construction in horizontal lifts even without the use of fabric; however, the influence of the fabric may also be enhanced by the technique, particularly for fabrics with relatively high modulus. The use of the outside-inside construction technique is recommended for all geotextile reinforced embankments.

9) The construction of embankments on peat deposits is often facilitated by the presence of a root mat overlying the deposit. Provided failure does not occur, this root mat will often be sufficiently stiff to govern the embankment performance and only a very high modulus geotextile would directly influence embankment response. However, increasing embankment height tends to cause tensile failure within the root mat. The presence of a geotextile of even moderate modulus will reduce the likelihood of root mat splitting beneath the embankment. Under these circumstances, the primary function of the geotextile is to maintain the integrity of the root mat. Again, the modulus of the fabric is the major characteristic governing the effectiveness of the geotextile for this purpose. Increasing fabric modulus reduces root mat tension near the embankment centreline but may in fact increase root mat tension outside the reinforced zone, causing tension cracks.

Table 5.2/ Required Degree of Fabric Survivability as a Function of Subgrade Conditions and Construction Equipment Techniques [slightly modified from Haliburton (1981)]

Subgrade Conditions	Construction Equipment/Techniques	
	Low Ground Pressure Equipment. No pre-rutting, etc. (≤ 20 kPa)	Medium to High Ground Pressure Equipment. Pre-rutting, etc. (20-40 kPa)
Subgrade has been cleared of all obstacles except grass, weeds, leaves, and fine wood debris. Surface is smooth and level, such that any shallow depressions and humps do not exceed 150 mm in depth and height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Low	Moderate
Subgrade has been cleared of obstacles larger than small- to moderate-sized tree limbs and rocks. Tree trunks should be removed or covered with a partial working table. Depressions and humps should not exceed 450 mm in depth and height. Larger depressions should be filled.	Moderate	High
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than 150 mm above subgrade. Fabric may be draped directly over the tree trunks, stumps, large depressions and humps, holes, stream channels, and large boulders. Items should be removed if placing the fabric and cover material over them will then significantly distort fabric.	High	Very High

Although it is well recognized that a root mat may significantly increase stability and reduce settlement, there are many practical problems associated with establishing the depth and characteristics of a root mat. In particular, the beneficial effects of the root mat should be seriously considered only if the designer is convinced of the general competence of the root mat, since any localized weak spots may negate these benefits.

10) When used as reinforcement, geotextiles are wholly contained within the embankment. They cannot, therefore, be expected to perform the same function in "floating" an embankment across a soft underlying deposit as a competent root mat (which is "anchored" by the roots from trees outside the embankment).

5.6.1/ Design of Low Embankments on Fibrous Peat

The behaviour of embankments on fibrous peat is characterized by significant partial dissipation of excess pore pressures at normal construction rates, together with large end of construction settlements. These two factors make the use of conventional stability analyses quite inappropriate even for unreinforced embankments and, in fact, there is no simple way of calculating the stability of embankments on fibrous peat. Nevertheless, shear deformations arising from partial or complete plastic failure in the peat may be a significant factor affecting embankment performance. Consequently, an objective of the embankment design should be to limit or minimize these shear deformations.

The finite element procedure adopted in this study does permit consideration of these factors but is not suitable for routine design. Thus, to provide some guidance for the selection of geotextiles, a parametric study was performed for a granular embankment ($\phi \geq 32°$) on a peat deposit with properties given in Table 5.4. Consideration was also given to the possibility that this peat was underlain by a very soft clayey soil with properties given in Table 5.5. While it cannot

Table 5.4/ Peat Parameters Adopted

c'	1.8 kPa
ϕ'	27°
ψ	0°
υ'	0.15
K_o'	0.176
G_s	1.5
e_o	9
γ	10.3 kN/m^3
E' ($0 \leq \sigma_v' \leq 20$ kPa)	85 kPa
E' ($20 < \sigma_v' \leq 40$ kPa)	110 kPa
E' ($40 \leq \sigma_v' \leq 60$ kPa)	140 kPa
E' ($\sigma_v' > 60$ kPa)	225 kPa

Table 5.5/ Clay Soil Directly Beneath the Embankment

c_u	variable (3.85, 5, 5.77, 7.5, 10 kPa)
ϕ_u	0°
υ'	0.48
K_o	0.5
G_s	2.7
e_o	variable (3.5, 5.7)
γ_T	variable (13.5, 12.3 kN/m^3)
E_u	variable (500, 1000 kPa)

Table 5.3/ Minimum Fabric Properties Required for Fabric Survivability [slightly modified from Haliburton (1981)]

Required Degree of Fabric Survivability	Ultimate Tensile Strength[1] (kN/m)	Secant Modulus at 10% Strain (kN/m)	Puncture Strength[2] (N)	Burst Strength[3] (kPa)	Abrasion Resistance Residual Ultimate Tensile Strength[4] (%)
Very High	50	175	900	4150	30
High	35	125	550	3500	30
Moderate	10	50	450	2100	20
Low	5	20	325	1350	20

Notes:
1 Values are for the strongest fabric direction, wide strip tensile test method.
2 ASTM D-751-68 test method, steel ball replaced by round-end steel cylinder.
3 Mullen burst strength, ASTM D-751-68 test method.
4 ASTM D-1175-64T abrasion resistance test method, rotary platform, double head method with rubber base CS-17 "Calibrase" abrasive wheels; 1000 g load on each wheel, 1000 revolutions. Residual ultimate tensile strength obtained by a wide strip tensile test method and expressed as a percent of the unabraded ultimate tensile strength; applicable for machine and cross directions.

be expected that all peats will have the same properties as those assumed here, it would be unusual for a fibrous peat to have a combination of parameters more critical than those assumed (although this possibility cannot be excluded). Clearly, if there is any doubt for a particular peat, this doubt can be resolved by performing appropriate tests as described in Chapter 3.

The guidelines for the selection of geotextile reinforcement are summarized in Table 5.6 for three depths of peat (3, 5 and 8 m). Figure 5.1 gives a design chart which can be used for estimating the geotextile modulus required to provide reasonable performance for an embankment constructed to height h (above the original ground level) on peat which is underlain by a firm base. It is assumed that the geotextile is located at, or close to, the surface of the peat and that the geotextile-soil interface friction angle exceeds 25° (in many cases an angle of 20° would be adequate but there are situations where some interface slip might be expected for an angle of 20°). These recommendations are based on the additional assumptions that:

a) there is no competent root mat, and
b) the maximum excess pore pressure is 0.34 times the change in total stress, σ_v, due to embankment construction.

Clearly, the presence of a competent root mat will improve embankment performance. The excess pore pressure will depend on the rate of construction and can be controlled. If there is any question of the excess pore pressures exceeding that assumed, then instrumentation should be provided and the construction rate controlled to ensure that the maximum excess pore pressure does not exceed 0.34 σ_v.

Figure 5.1/ Design Chart for Peat Underlain by a Firm Base (see text for assumptions and limitations)

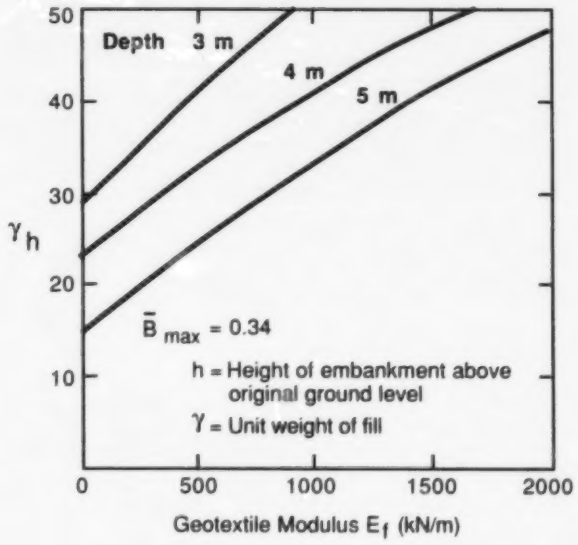


Table 5.6/ Recommended Geotextile Modulus Values for Embankments on Fibrous Peat (see text for limitations)

Peat Thickness (m)	Underlying Strata	Strength of Underlying Strata c_u (kPa)	Maximum Height of Fill above Original Ground Level (m) 1.0	1.5	2.0	2.5
3	Firm	—	NRR[1]	NRR	500 (5%)[2]	1000 (6%)
3	2 m clay	15	NRR	NRR	500 (5%)	1000 (6%)
		10	NRR	NRR	500 (6%)	1000 (6.5%)
3	2 m clay	7.5	NRR	150 (14%)	500 (8%)	1000 (7%)
		5	NRR	150 (10%) - 500 (8%)[4]	2000 (4%)	PF[3]
3	2 m clay	2.5-5	150 (10%)	2000 (5.5%)	PF	PF
5	Firm	—	NRR	150 (14%) - 500 (6.5%)	500 (9.5%) - 1000 (5.5%)	1000 (6.5%) - 2000 (4%)
5	3 m clay	15	NRR	150 (14%) - 500 (6.5%)	500 (9.5%) - 1000 (5.5%)	1000 (6.5%) - 2000 (4%)
		10	150 (5%)	500 (8.5%) - 1000 (7.5%)	2000 (5.5%)	PF
5	3 m clay	7.5	500 (5%)	1000 (8.5%) - 2000 (5%)	PF	PF
8	Firm	—	350 (15%) - 1000 (7%)	500 (14%) - 1000 (8%)	1000 (9%) - 2000 (6%)	2000 (6%)

Notes:
1. NRR = No reinforcing geotextile required.
2. 500 (5%) = A geotextile with modulus E_f = 500 kN/m is recommended. Under the assumed conditions, a maximum geotextile strain of approximately 5% is anticipated.
3. PF = Potential failure for the assumed conditions: Do not construct.
4. 150 (14%) - 500 (8%) = For the assumed conditions, the embankment could be constructed using E_f = 150 kN/m but the expected strain is large. If conditions are likely to be as assumed, a higher modulus geotextile is recommended. E_f = 500 kN/m would give 8% strain under the assumed conditions.

In some cases (see Table 5.6) a range of modulus values has been recommended. If the anticipated field conditions are as bad as those assumed, then a geotextile should be selected to have a modulus in the upper end of the range. If there is a competent root mat and/or the anticipated maximum excess pore pressure is less than 0.25, then a fabric with a modulus in the lower end of the range should be adequate.

The recommendations in Table 5.6 assume the embankment is constructed of conventional fill. If lightweight fill is available, then its use should be seriously considered. Table 5.7 gives recommended geotextile properties and lightweight fill (sawdust) thickness for a 5 m thick peat deposit and these provide an alternative design to that given in Table 5.6. The thickness of sawdust given in Table 5.7 corresponds to the compacted thickness and was selected to ensure that the sawdust would be submerged (assuming a water table at the surface) at the end of construction. For peat deposits with depths or compressibility slightly different from that assumed, use the same geotextile as suggested in Table 5.7 but adjust the thickness of the sawdust so it is approximately equal to the anticipated settlement.

The numbers given in brackets in Tables 5.6 and 5.7 represent the expected geotextile strains for the given geotextile for the worst assumed conditions. The expected force in the geotextile can be deduced from the strain and the modulus. Since the geotextile strains are sensitive to construction sequence, strains greater than or less than those indicated in Tables 5.6 and 5.7 may be anticipated under some circumstances and any geotextile selected should have an adequate factor of safety (preferably a factor of 2) against failure of the geotextile itself. Thus, a geotextile should be selected which:

a) has an appropriate modulus, and
b) has an adequate factor of safety against failure of the geotextile.

Figure 5.2/ Variation in Vertical Strain with Change in Effective Stress Based on Field Observations

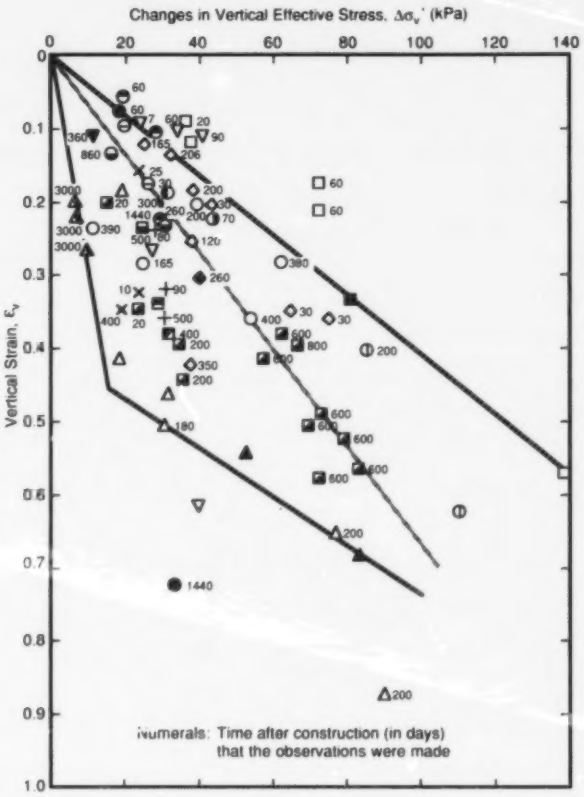


Table 5.7/ Recommended Geotextile Modulus Values for Embankments Constructed on Fibrous Peat Using Lightweight Fill (see text for limitations)

Peat Thickness (m)	Underlying Strata	Strength of Underlying Strata c_u (kPa)	Maximum Height of Fill above Original Ground Level (m)			
			1.0	1.5	2.0	2.5
5	Firm	—	NRR LF = 1 m	NRR* LF = 1.5 m**	150 (105%) - 500 (6%)*** LF = 1.9 m	500 (9%) LF = 2.1 m
5	3 m clay	15	NRR LF = 1 m	NRR LF = 1.5 m	150 (10%) - 500 (6%) LF = 1.9 m	500 (9%) LF = 2.1 m
5	3 m clay	7.5	NRR LF = 1 m	150 (14%) LF = 1.5 m	500 (6%) LF = 1.9 m	1000 (9%) LF = 2.5 m
5	3 m clay	2.5 - 7.5 LF = 1.2 m	500 (5%) LF = 1.6 m	1000 (5%) LF = 2.2 m	2000 (4%) LF = 2.5 m	2000 (6%)

Notes:
* NRR =No reinforcing geotextile required.
** LF = Thickness of lightweight fill used. For any particular application, the thickness of lightweight fill should be selected to be approximately equal to the end of construction settlement.
*** For the assumed conditions, the embankment could be constructed using E_f = 150 kN/m but the expected strain would be 10%. A higher modulus fabric of E_f = 500 kN/m would give better performance with expected strain of 6%.

The height above original ground level, h, used in the design should be the maximum height of the embankment and includes any surcharge which may be applied.

Where practical, surcharging is the recommended means of improving long-term performance of embankments on peat. Normally consolidated peat deposits may be expected to experience significant long-term settlement. These long-term deformations can be greatly reduced by applying a surcharge to the peat. The use of a geotextile may permit the use of a surcharge in situations where the additional height of fill might otherwise have caused collapse or excessive shear deformations. A surcharge of between 0.5 and 1 m is recommended. The surcharge should be left on as long as practicable. When a surcharge is used, the surcharge should be used over the entire length of the peat deposit. (Use of a surcharge over only a portion of the deposit may be expected to result in uneven settlement and poor long-term riding characteristics.)

The primary settlement (i.e., excluding the secondary consolidation) of embankments designed in accordance with the foregoing guidelines may be estimated from the results of 1-D consolidation tests on representative samples of peat by calculating the vertical strain, ε_i, of layers of peat from the equation:

$$\varepsilon_i = \frac{\Delta e}{1 + e_o}$$

where:

- e_o = the initial void ratio
- Δe = the change in void ratio measured from a plot of e *vs* σ' for the stress range from σ_o' to $\sigma_o' + \Delta\sigma'$
- σ_o' = the estimated initial stress (if in doubt, assume it is zero)
- $\Delta\sigma'$ = the anticipated change in effective stress $= \gamma h + \gamma_s' S$
- γ = the unit weight of the fill above original ground
- γ_s' = the submerged unit weight of the fill below original ground level (assuming the water table is at the surface)
- h = maximum embankment height above original ground level
- S = the estimated settlement $= \Sigma \varepsilon H_i$
- H_i = initial thickness of layer i

Clearly, the determination of the settlement will involve an iterative procedure (unless sawdust of thickness S is used) since the change in effective stress depends on the settlement S.

An alternative method of estimating settlement (or providing an initial estimate for the calculation above) is to use empirical relationships such as shown in Figure 5.2. This figure summarizes the settlement of a large number of embankments on peat. The settlements are as reported in the literature and they correspond to different times after construction (i.e., they incorporate different amounts of secondary compression). The numeral after each data point indicates the number of days after construction that the settlement was reported. The embankments where very large strains were reported usually involved large shear deformations. Design in accordance with the foregoing recommendations should limit the shear deformations and the settlement of these embankments.

5.6.2/ Design of Low Embankments on Cohesive Soil

The behaviour of embankments constructed on very soft cohesive soil where there is negligible excess pore pressure dissipation during construction will be governed by the undrained shear strength and undrained modulus of the clay, and the depth of the deposit. The effect of a geotextile will depend on all three factors, together with the height and unit weight of the embankment fill.

5.6.2.1/ Collapse of a Reinforced Embankment

The collapse height of an unreinforced embankment corresponds to the height at which the shear strength is fully mobilized along a potential rupture surface. For a reinforced embankment, however, if the potential failure surface is intersected by the reinforcement, then collapse will involve failure of both the soil and some aspect of the soil-reinforcement system. Clearly, failure of the soil-reinforcement system may take the form of a failure of the soil-geotextile interface (i.e., slip of the geotextile relative to the soil) or a failure of the geotextile. However, if the geotextile permits large deformations prior to reaching failure strain (e.g., low modulus geotextiles), or if the deposit is deep, then collapse of the reinforced embankment may occur before failure of the geotextile or the geotextile interface. Prior to actual failure of the geotextile or the geotextile-soil interface, the contribution of the geotextile to the embankment stability is governed by a condition of strain compatibility and the relative moduli of the soil and geotextile.

If an embankment were reinforced with a geotextile having a modulus so small that negligible load is developed in the geotextile at collapse, one would expect the strains just prior to collapse, and the collapse height of the reinforced embankment, should be the same as those corresponding to failure of a similar unreinforced embankment. The strain that would occur in the geotextile just prior to collapse will be referred to as the "allowable compatible strain".

In the design of reinforced embankments, it is considered that failure should be deemed to have occurred if the shear strength of the soil is fully mobilized along a potential rupture surface and any of the following conditions are satisfied:

1) The potential rupture surface is not intersected by the geotextile. This allows for failure of the entire embankment as a rigid body. This mechanism may be critical for reinforced embankments on deep soil deposits. Thus, for a homogeneous deposit with depth greater than 0.84 times the embankment width, the critical height is given by the Prandtl (1920) solution, irrespective of how much reinforcement is used, i.e., γh (collapse) = 5.14 c_u.

2) The tensile strength of the fabric is reached. This condition will rarely govern, but should still be checked.

3) Pullout of the fabric from the soil occurs (i.e., the soil-fabric interface shear strength is developed above and below the fabric on one side of the potential rupture surface).

4) Shear slip occurs between the geotextile and underlying soft clayey soil. In this case, failure involves the soil squeezing out from between the geotextile and the base. This will often be the controlling failure mechanism for embankments reinforced with a high modulus geotextile and resting on deposits of very soft soil.

5) The lateral deformation of the embankment exceeds that of an unreinforced embankment just prior to collapse (i.e., the strain in the geotextile reaches the "allowable compatible strain"). This criterion will usually govern when low modulus fabrics are used.

Item (5) is intended to reduce the likelihood of collapse of the embankment due to post peat strength loss and creep which may be expected at the large strains permitted by low modulus fabrics prior to fabric or fabric-soil interface failure.

5.6.2.2/ Design Charts

Adopting the most critical failure condition (for 1-5 above) for each case, a series of design charts (Figures 5.3 to 5.6) were developed from the finite element analysis described in Chapter 4. In the development of these charts it has been assumed that:

- the embankment has a crest width of 13.4 m and 2:1 side slopes,
- the fill material has a friction angle of 32°,
- the geotextile is located close to or on the foundation soil,
- the geotextile-foundation interface strength is equal to 0.77 c_u of the foundation (i.e., a factor of safety of 1.3 is applied to the undrained strength of the foundation), and
- the embankment is to be designed with a factor of safety of 1.3.

The design charts are given for two depths (3 m and 5 m), two values of undrained modulus (500 and 1000 kPa), and a range of undrained shear strengths (5, 7.5 and 10 kPa). The procedure for the use of these charts is illustrated below for the case of a proposed embankment 2 m high (above original ground level) to be constructed on a 3 m thick clayey foundation with c_u = 7.5 kPa and E_u = 500 kPa. The embankment is to be 13.4 m wide with 2:1 side slopes and is to be constructed of granular fill with $\phi > 32°$.

Step 1: Estimate the required embankment height, h, above original ground level at the end of construction. This height may be above the final required grade to allow for settlement or, alternatively, this may be the final grade level and the embankment will have to be brought back to grade at some later time after settlement has occurred. For purposes of the example, it is assumed that h = 2 m.

Step 2: Estimate the unit weight, γ, of the fill and hence the quantity, γh. (In the example, take γ = 19 kN/m^3, thus γh = 19 x 2 = 38 kPa.)

Step 3: Select the appropriate chart from Figures 5.3 and 5.5 for the given soil depth, D, and undrained modulus, E_u. As shown in Figure 5.3, the design chart is entered for the known value of γh, moving horizontally until the dashed curve is reached for the required value of c_u, and then downwards to read off the geotextile modulus required for a factor of safety of 1.3. (For the example case, the required modulus is 500 kN/m.)

Step 4: Estimate the force in the geotextile which would be mobilized under ultimate conditions. This force may be estimated by selecting the appropriate chart from Figures 5.4 and 5.6 for the given values of D and E_u, then enter the design chart with the known geotextile modulus from Step 3, move upwards until the curve for the appropriate design value of c_u is reached, then move horizontally and read off the ultimate geotextile force as illustrated in Figure 5.4. (In the example, the ultimate geotextile force is 20 kN/m.)

Step 5: Select a geotextile which has the required modulus (or higher) and an appropriate factor of safety against geotextile failure. If a geotextile is selected with a modulus E_{fa} in excess of the required value E_f from Step 3, the ultimate geotextile force should be increased to:

$$T(\text{ultimate}) = (E_{fa} \div E_f) \times T(\text{ultimate determined from Step 4 above})$$

In the example, a modulus of 500 kN/m is required and the ultimate geotextile force (Step 4) was 20 kN/m; thus, the estimated geotextile strain at ultimate is 20/500 = 0.04. An inspection of Table 5.1 would suggest that Tensar SR2 may be a suitable geotextile. Allowing for a modulus of approximately 630 kN/m over the strain range 0-4% (obtained by interpolation between the secant modulus values given for 0-2% and 0-5%), the adjusted ultimate fabric force is 630/500 x 20 = 25 kN/m. Hence, the factor of safety against geotextile failure is:

FS = 60(From Table 6.1)/25 = 2.4

which is considered adequate.

The design charts presented in Figures 5.3 to 5.6 can be used for the specific cases considered and, with some judgement and interpolation, they could also be used for intermediate cases. However, based on the finite element study, an alternative modified limit equilibrium method has been developed which, although approximate, should allow more convenient interpolation for cases not explicitly considered in the design charts.

5.6.2.3/ Modified Limit Equilibrium Approach

The most significant advance of the proposed method of analysis over earlier extended limit equilibrium techniques is the estimation of the allowable geotextile strain by rational means.

An extensive finite element study was performed to determine the allowable compatible strain for a variety of cases (e.g., see Chapter 4). Since the allowable compatible strain is the maximum strain in a geotextile of negligible modulus just prior to collapse, it will depend on the compatible deformations of the soil which occur prior to collapse. These deformations are a function of the width of the embankment, B, (see Figure 5.7), the depth of the soft soil beneath the embankment, D, the undrained modulus of the soil, E_u, the undrained shear strength, c_u, the unit weight of the fill, γ_f, and the height of the embankment at collapse, H_c.

It was found that the effect of these parameters could be represented in terms of a single non-dimensional parameter Ω defined as:

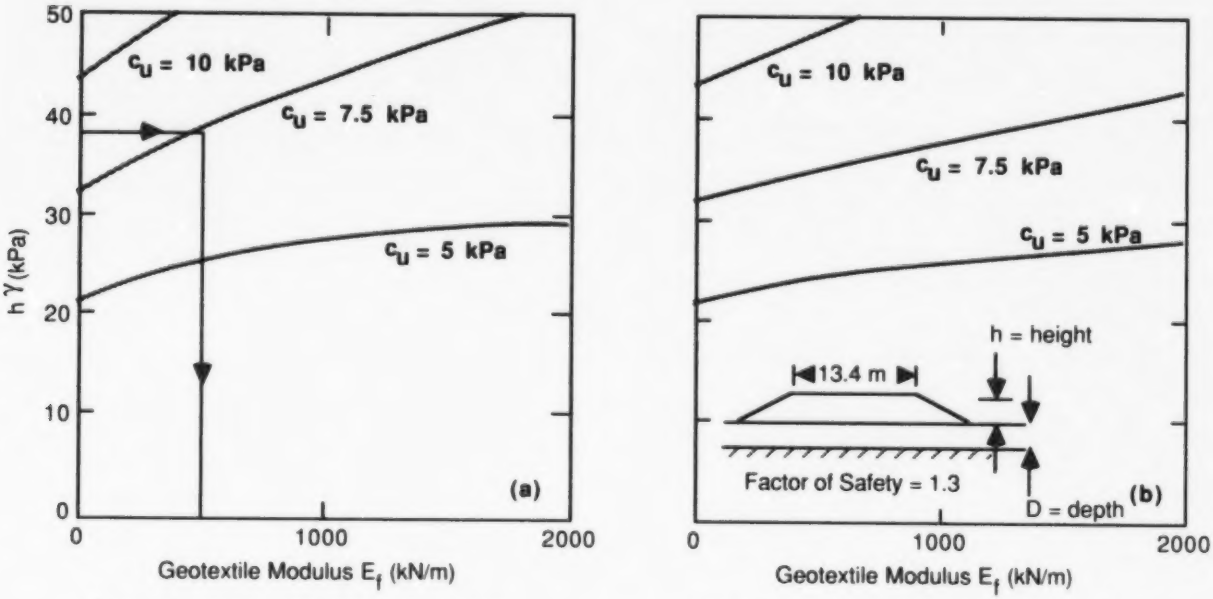
$$\Omega = \left(\frac{\gamma_f H_c}{c_u}\right)\left(\frac{c_u}{E_u}\right)\left(\frac{D}{B}\right)_e^2 \qquad (5.1)$$

where: γ_f, H_c, E_u, c_u, D and B are as above and:

$(D/B)_e = 0.2$ for $D/B < 0.2$
$(D/B)_e = D/B$ for $0.2 \le D/B \le 0.42$
$(D/B)_e = 0.84 - D/B$ for $0.42 < D/B \le 0.84$
$(D/B)_e = 0$ for $0.84 < D/B$

$(D/B)_e$ being the effective depth-to-crest width ratio of the deposit (to be discussed shortly).

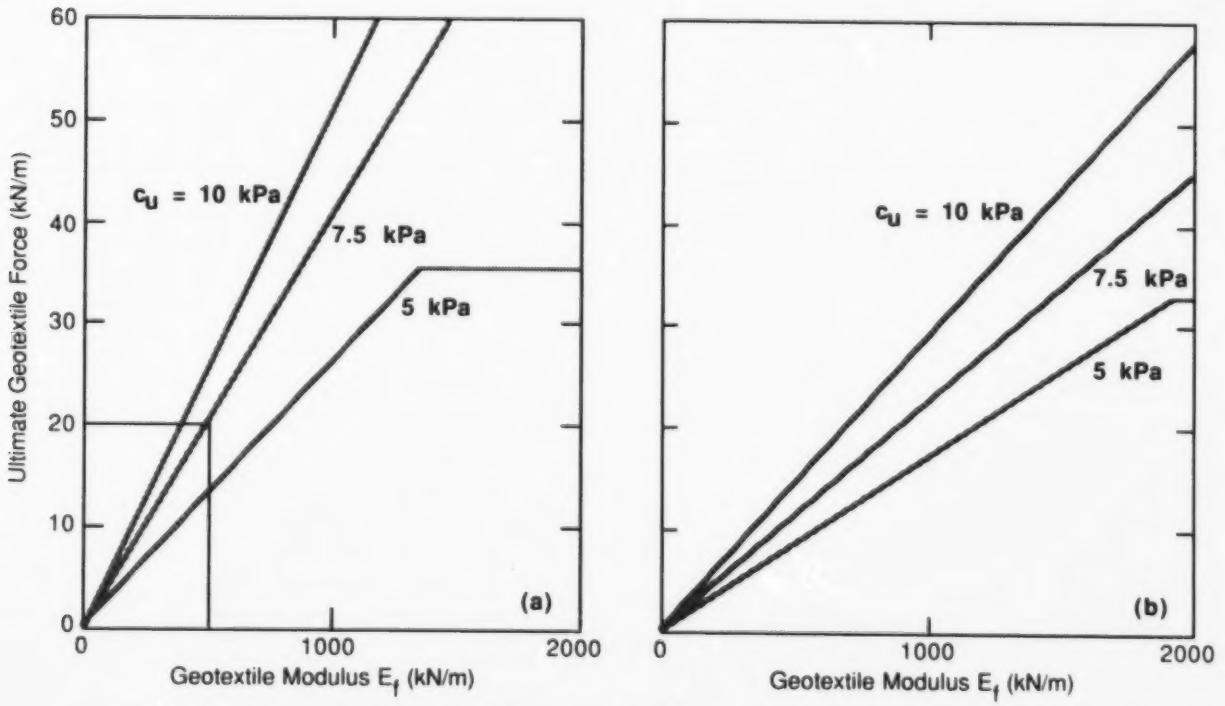
Figure 5.3/ Allowable Embankment Height, h; D = 3 m



(a) D = 3 m, E_u = 500 kPa, υ_u = 0.48

(b) D = 3 m, E_u = 1000 kPa, υ_u = 0.48

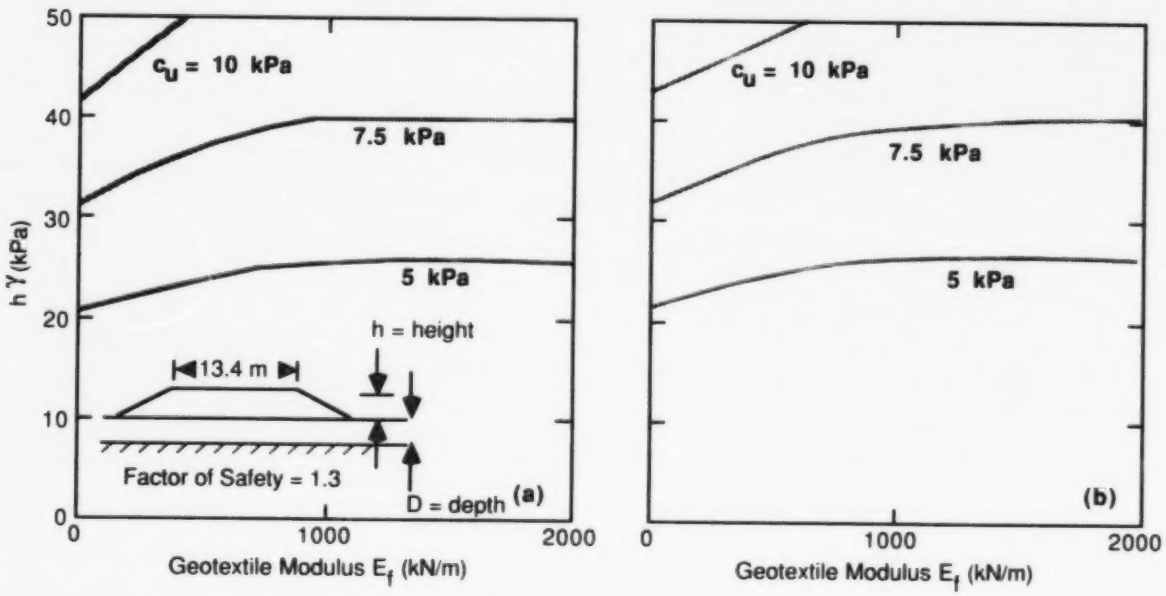
Figure 5.4/ Force Developed in the Geotextile under Ultimate Conditions, h; D = 3 m



(a) D = 3 m, E_u = 500 kPa, υ_u = 0.48

(b) D = 3 m, E_u = 1000 kPa, υ_u = 0.48

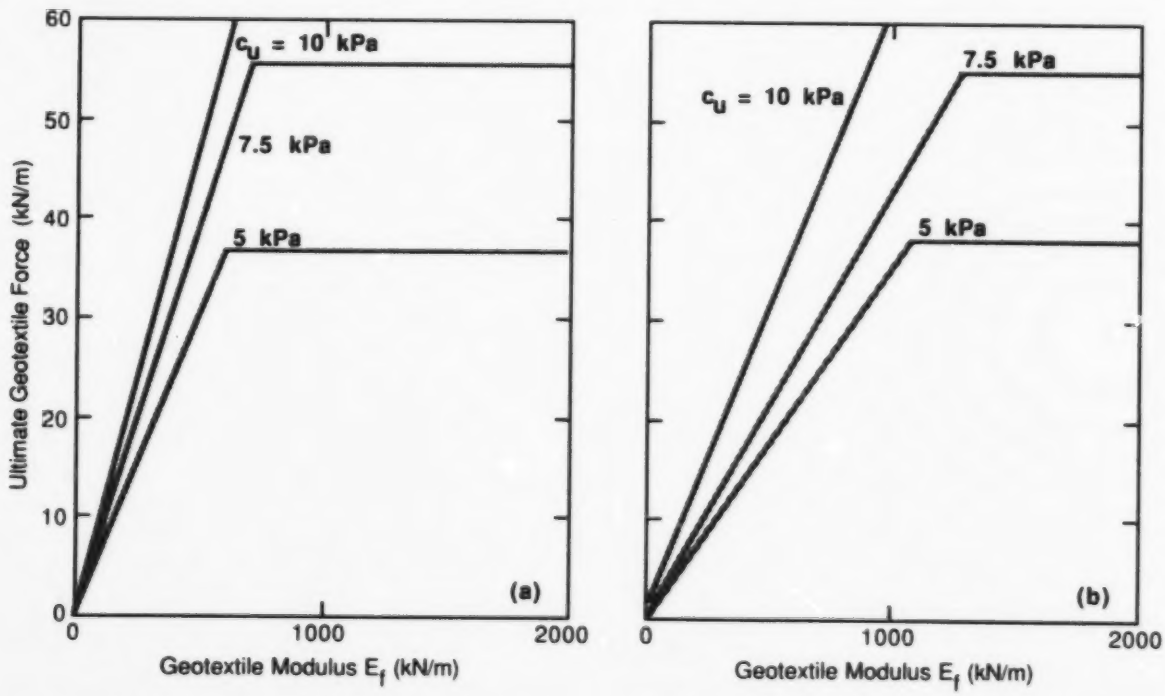
Figure 5.5/ Allowable Embankment Height, h; D = 5 m



(a) D = 5 m, E_u = 500 kPa, υ_u = 0.48

(b) D = 5 m, E_u = 1000 kPa, υ_u = 0.48

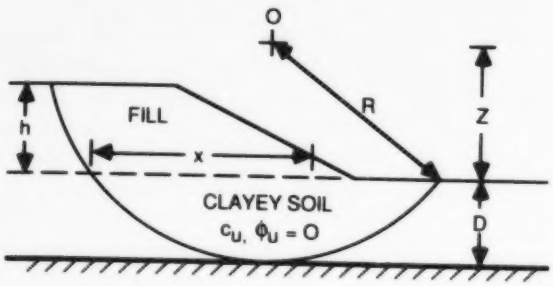
Figure 5.6/ Force Developed in the Geotextile under Ultimate Conditions, h; D = 5 m



(a) D = 5 m, E_u = 500 kPa, υ_u = 0.48

(b) D = 5 m, E_u = 1000 kPa, υ_u = 0.48

Figure 5.7/ Notation Used in Modified Limit Equilbrium Analysis



SYMBOLS

- D = depth of soft clayey foundation
- B = width of the embankment crest
- h = height of the embankment above original ground level
- R = radius of trial slip circle
- z = vertical distance from the centre of the trial slip circle to the geotextile
- x = horizontal distance from the point of intersection of the trial slip circle with geotextile to the edge of the geotextile
- c_u = undrained shear strength of the foundation material
- E_u = undrained Young's modulus of the foundation material
- γ = unit weight of the foundation material
- c_f = cohesion intercept for the fill
- ϕ_f = angle of internal friction of the fill
- γ_f = unit weight of the fill
- E_f = modulus of the geotextile determined over the appropriate range of strain (in kN/m)
- T_u = ultimate tensile capacity of the geotextile (in kN/m)
- F = factor of safety against embankment failure

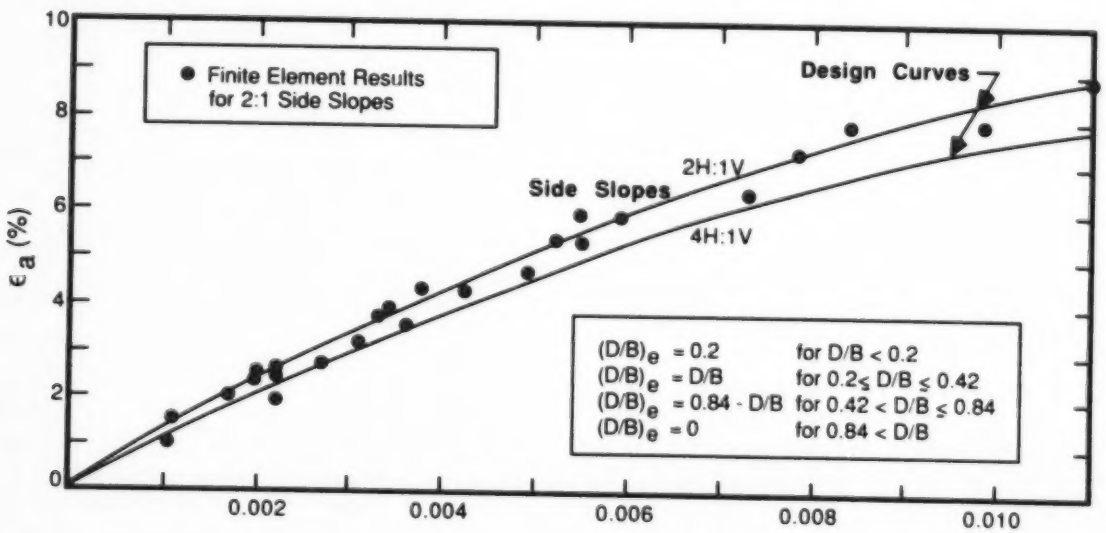
The allowable compatible strain may be directly related to the parameter as shown in Figure 5.8. It is suggested that in a stability analysis of a reinforced embankment, the force in the fabric T (which contributed to the restoring moment) should not exceed that developed at the allowable compatible strain (i.e., $T = \varepsilon_a \cdot E_f$ where E_f is the secant modulus of the fabric of the strain range from 0 to ε_a).

For a given soil deposit and embankment width B, there is a critical height H_c which represents the maximum height to which an unreinforced embankment could be constructed. In many cases, the use of geotextile reinforcement will allow construction of the embankment to a height exceeding H_c. However, irrespective of the actual height, it is recommended that the horizontal deformations (and hence the allowable compatible strain ε_a) should not be permitted to exceed the values which would be expected just prior to collapse for an unreinforced (or very lightly reinforced) embankment. Thus, the allowable strain ε_a depends on the critical height H_c and not the actual height of the embankment h (i.e., the allowable height h of a reinforced embankment will be limited so that the lateral deformation will not exceed what would be expected for an unreinforced embankment of height H_c just prior to collapse).

The effect of a geotextile upon the stability of an embankment depends on the depth D of the deposit relative to the crest width B. For shallow deposits where D/B is less than 0.2, the effective crest width is $B_e = 5D$. That is to say, the failure is controlled by the actual depth of the deposit and increasing the width of the embankment will not affect either the collapse height or allowable compatible strain (that is, H_c and ε_a for $D/B < D/B_e$ will be the same as that for $(D/B)_e = 0.2$).

For values of D/B in the range from 0.2 to 0.42, increasing the D/B ratio increases the strains in the geotextile prior to collapse (all other things being equal) and this increases the contribution of the geotextile to embankment stability.

Figure 5.8/ Allowable Compatible Strain, ε_a, versus Dimensionless Ω



$$\Omega = \left(\frac{\gamma H_c}{c_u}\right)\left(\frac{c_u}{E_u}\right)\left(\frac{D}{B}\right)^2_e$$

However, there is an optimal D/B ratio of approximately 0.42 where the allowable compatible strain is the greatest.

For values of D/B greater than 0.42, increasing D/B actually leads to a reduction in the allowable strain. Figures 5.9 and 5.10 show the relative deformations of the soil, at collapse, obtained from finite element analyses for D/B = 0.33 and 0.55. An examination of the displacement components at the interface between the embankment and foundation for the two cases reveals that for D/B = 0.33, the component of the horizontal displacement is substantially greater than for D/B = 0.55. This horizontal displacement is responsible for mobilizing the forces in the geotextile. Thus, inclusion of a geotextile may be expected to have less effect on the stability of an embankment for D = 0.55B than for D = 0.33B (all other things, including B, being constant).

Figure 5.11 illustrates the variation in the allowable compatible strain with the depth of the deposits for fixed values of B, c_u, E_u and γ_f (note, however, that H_c does vary with layer depth and is not constant). It can be seen that for this case the peak allowable compatible strain occurs at a depth of approximately 5.7 m (D/B = 0.42) and the allowable compatible strain decreases for depths in excess of 5.7 m.

It can be shown that for a deep soil deposit (D/B > 0.84) the geotextile has no effect whatsoever on deep-seated stability (although it may still improve local stability near the edge of the embankment). This corresponds to the well established fact that the bearing capacity of a rough rigid footing on a deep uniform cohesive layer is identical to that of a smooth flexible footing (see Hill, 1950) and, neglecting the embankment strength, is given by $\gamma_f H_c = 5.14\ c_u$.

It was found numerically that the effect of increasing depth for D/B ratios in the range 0.42 to 0.84 could be approximately represented by defining Ω in terms of an effective ratio $(D/B)_e$ defined by:

$$(D/B)_e = 0.84 - D/B$$

This will give the maximum compatible strain ε_a for D/B = 0.42 and will give a decrease in Ω (and hence ε_a) for D/B > 0.42. For D/B < 0.84, the allowable strain is zero (i.e., $(D/B)_e = 0$ in Equation 5.1).

The relationship between ε_a and Ω given in Figure 5.8 represents an envelope to the finite element results obtained for B = 13.4 m and D in the range for 3 to 8.8 m. The actual data points for this case and 2:1 side slope are shown in the figure. The curve was also checked by performing additional analyses for width B in the range for 10 m to 30 m and depth up to 15 m and it was found that the curves given in Figure 5.8 provided a very good representation for D/B ratios less than 0.42 and was a reasonable envelope (i.e., it was conservative) for D/B ratios greater than 0.42.

It has previously been found (e.g., Rowe, 1982) that the strains developed in a geotextile depend somewhat on the construction sequence which is adopted (e.g., an outside-inside construction as compared to full width construction in horizontal lifts). The curves given in Figure 5.8 were checked by performing analyses for alternative construction sequences and it was found that estimates of stability based on allowable strain deduced from Figure 5.8 would be conservative in each case. To reduce the likelihood of failure of the fabric due to additional construction-induced strains, the allowable compatible strain should be the minimum of the value deduced from Figure 5.8 and 60% of the failure strain of the geotextile, where the failure strain is determined from a wide strip test on the particular geotextile.

Figure 5.9/ Velocity Field at Collapse D/B = 0.33

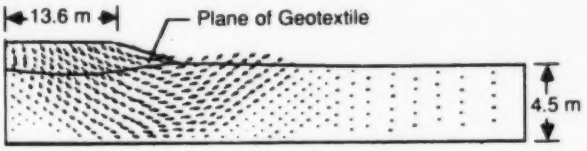


Figure 5.10/ Velocity Field at Collapse D/B = 0.55

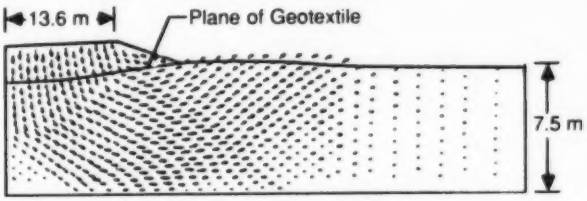
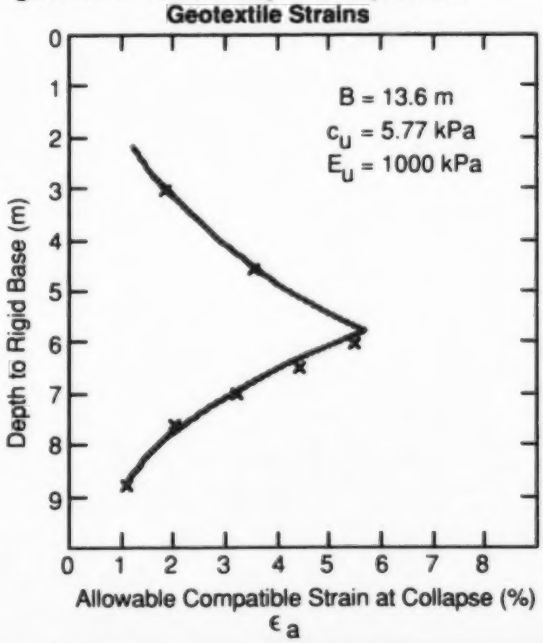


Figure 5.11/ Effect of Depth of Deposit on Geotextile Strains



In general, the greater the fabric strain mobilized prior to collapse of the embankment, the greater will be the contribution of the force mobilized in the geotextile to embankment stability. The magnitude of the strain developed in the geotextile will depend on the deformation of the soil which, in turn, depends on the undrained modulus of the soil. The effect of the modulus is reflected in the parameter Ω (see Equation 5.1) and it can be seen that the lower the modulus, the larger will be Ω, and hence (from Figure 5.8) the larger will be the allowable compatible strain. Thus, it is not conservative to underestimate the undrained modulus of the soil since this would imply larger allowable fabric strains (and a higher fabric force) than may actually be mobilized in the embankment prior to collapse.

Procedure

The procedure is an extension of the simplified Bishop slip circle analysis (Bishop, 1954). Hence, the conventional assumptions of the Bishop method apply to the extended approach. The additional assumptions necessary to include the effect of geotextile reinforcement are:

- The strains are small and the foundation response is "undrained".

- Any forces due to the geotextile only act to increase the restoring moment, they do not affect the Bishop slice normal forces.

- The geotextile force acts at the point of intersection of the geotextile and the trial slip circle.

- The geotextile is located at, or near, the foundation-fill interface.

- Referring to Figure 5.7 and defining:
  R = radius of the trial circle; and
  z = vertical distance from the centre of the trial circle to the plane of the geotextile, then the moment arm for calculating the restoring moment due to the geotextile force is taken to be:
  moment arm = (R+z)/2 for D/B ≤ 0.42
  moment arm = z for D/B > 0.42

  (Note that a moment arm of z is conservative for all D/B but may be a little too conservative for D/B < 0.42.)

- The magnitude of the geotextile force, T, used to calculate the restoring moment is the lesser of:

  a) The Allowable Geotextile Force, T_a — This is determined from consideration of the ultimate tensile strength of the geotextile and its creep properties. Generally, T_a should not exceed 60% of the ultimate capacity of the geotextile T_u determined from a wide strip tension test at a strain rate of 2% per minute. (For some polypropylene- and polyethylene-based geotextile materials, a value of $T_a \leq 0.5\ T_u$ may be necessary.)

  b) The maximum bond force that can be developed between the geotextile and the soft foundation — Referring to Figure 5.7, this limit is given by $T_b = xC_s$, where x may not exceed half the embankment base width (B/2 + tH).

  c) The force mobilized in a geotextile of modulus E_f at an allowable compatible strain, ε_a (i.e., $T_c = \varepsilon_a \cdot E_f$) — As noted in the previous section, the allowable strain will depend on the geometry and soil properties. The effect of these factors may be combined in terms of the dimensionless parameter Ω where Ω is defined by Equation 5.1, viz.:

$$\Omega = \left(\frac{\gamma_f H_c}{c_u}\right)\left(\frac{c_u}{E_u}\right)\left(\frac{D}{B}\right)^2_e$$

  The relationship between ε_a and Ω for a single layer of geotextile is given in Figure 5.8 for two different embankment slopes.

- The expression for factor of safety has been modified as follows to include the effect of geotextile reinforcement.

$$F = \frac{\Sigma \text{B.R.M.} + \text{R.R.M.}}{\Sigma \text{B.O.M.}}$$

  where: F = factor of safety for the reinforced embankment
  ΣB.R.M. = sum of the restoring moments (as in the unreinforced case)
  ΣB.O.M. = sum of the overturning moments (as in the unreinforced case)
  R.R.M. = restoring moment due to the reinforcement
  = T(R+z)/2 for D/B ≤ 0.42
  = Tz for D/B > 0.42

  Rearranging the expression for factor of safety yields:

$$\Sigma \text{B.O.M.} = \frac{\Sigma \text{B.R.M.} + \text{R.R.M.}}{F}$$

  Note the factor of safety, F, has been applied to both the soil restoring moment and the reinforcement restoring moment. A typical factor of safety is 1.3.

Application

The extended Bishop method of slices, just described, can be incorporated into a simple design procedure. The goal of the procedure is to select geotextile reinforcement which will provide an adequate factor of safety against undrained failure of an embankment on a soil which can be idealized as having a uniform strength. Thus, the selection of appropriate reinforcement involves ensuring that there is an adequate factor of safety against collapse due to reinforcement failure, reinforcement creep, reinforcement-soil interface failure, and excessive geotextile strain.

In this section, the proposed design procedure will be presented and its use illustrated by application to typical problems.

Illustrative Example 1

This example illustrates how a geotextile reinforcement may be selected to give the desired factor of safety, F, against rotational failure. The procedure to be adopted involves five steps.

Step 1: Problem Definition

Define the required parameters and geometry of the problem as indicated in Figure 5.7.

Design Parameters:

a) Geometry: h, D, B, t

b) Fill Parameters: c_f, ϕ_f, γ_f

c) Foundation Parameters: c_u, E_u, γ

d) Desired Factor of Safety: F

(Due to the application of the factor of safety, F, to the reinforcing restoring moment (in Equation 5.3), the factor of safety associated with the estimation of the mobilized reinforcing force is also F.)

To illustrate the procedure, consider a two-lane highway embankment with the following parameters:

h = 2 m, D = 5 m, B = 13.4 m, t = 2, c_f = 0, ϕ_f = 39°, γ_f = 21 kN/m³, c_u = 8 kPa, E_u = 1000 kPa, γ = 13.5 kN/m³, and a required factor of safety equal to 1.3.

An iterative procedure is necessary for the determination of the required geotextile properties. As an initial estimate, E_f = 1000 kN/m and T_a = 60 kN/m have been selected.

Step 2: Determine the Collapse Height, H_c, of an Unreinforced Embankment

a) Determine F*, where F* is the critical factor of safety for an unreinforced embankment of height h (h = design embankment height). If F* is greater than or equal to F, the required factor of safety, then no reinforcement is required and the procedure is terminated.

b) Calculate the collapse height, H_c.

$$H_c = F^* \cdot h$$

(This calculation is not strictly valid due to the slight nonlinearity in the relationship between F* and h; however, a more rigorous examination indicates that the approximation is adequate for most purposes.) For this example, the value of F* determined for h = 2 m is 1.04 and hence $H_c = F^* \cdot h$ = 2.08 m.

Step 3: Determine the Allowable Compatible Strain, ε_a

$$\text{Calculate } \Omega = \left[\frac{\gamma_f H_c}{c_u}\right]\left(\frac{c_u}{E_u}\right)\left(\frac{D}{B}\right)_e^2 = \left[\frac{\gamma_f H_c}{E_u}\right]\left(\frac{D}{B}\right)_e^2$$

Note that for $D/B \le 0.2$ $(D/B)_e = 0.2$

$0.2 < D/B \le 0.42$ $(D/B)_e = D/B$

$0.42 < D/B < 0.84$ $(D/B)_e = 0.84 - D/B$

$0.84 < D/B$ $(D/B)_e = 0$

b) With the calculated value of Ω and the given slope parameter t, select the corresponding value of ε_a from Figure 5.8.

For the example:

(a) D/B = 5/13.4 = 0.37, therefore $(D/B)_e$ = D/B = 0.37 and

$$\Omega = \left[\frac{\gamma_f H_c}{E_u}\right]\left(\frac{D}{B}\right)^2 = \frac{21\,(2.06)}{1000}\left(\frac{5}{13.5}\right)^2 = 0.0059$$

With Ω = 0.0059 and t = 2, the value of ε_a from Figure 5.8 is 5.9%.

Step 4: Determine the Factor of Safety, F_c, for the Reinforced Embankment

Calculate the critical factor of safety, F_c, for the reinforced embankment of height, h, including an additional restoring moment, R.R.M., viz.

$$\text{R.R.M.} = T(R + z)/2 \quad \text{for } D/B \le 0.42$$
$$= T \cdot z \quad \text{for } D/B > 0.42$$

where: T = minimum of (a) T_a

(b) $T_b = x \cdot c_s$

(c) $T_c = \varepsilon_a \cdot E_f$

(see Fig. 5.7 but where $x \le B/2 + th$)

This step requires a computer search for the critical slip circles. The critical circles for the reinforced and unreinforced cases are generally not the same.

For this example, the critical factor of safety is calculated to be 1.3 and the selection of geotextile forces is governed by the allowable compatible strain of 5.9%. For this case, the interface shear is assumed to be mobilized only along the bottom surface of the geotextile. This conservative assumption is necessary because of the expected differences in the stress strain behaviour of the top and bottom interfaces.

Step 5: Check the Performance of the Selected Reinforcement

If F_c = F, the performance of the selected geotextile is satisfactory. If interface slip controls the selection of T, one should reduce the selected geotextile modulus to $E_f = T_b/\varepsilon_a$ and repeat steps 4 and 5. This prevents overdesign of the reinforcement. If interface slip does not control the design, then the desired geotextile properties (E_f, T) have been determined and the solution is complete.

If F_c > F, select a fabric with lower E_f and/or T, depending on which property is overdesigned, and repeat steps 4 and 5.

If F_c < F, select a fabric with a higher E_f and/or T, depending on which property is deficient, and repeat steps 4 and 5. If interface slip governs the selection of T, the desired factor of safety cannot be achieved by inclusion of a single layer geotextile. A different design is required (e.g., flatter side slopes).

For this example, since the value of F_c = F = 1.3 after the first iteration, the performance of the selected reinforcement is adequate and the solution is terminated.

The selected geotextile properties are E_f = 1000 kN/m and T = 60 kN/m for an overall factor of safety of 1.3.

Illustrative Example 2

This second example illustrates how the factor of safety, F, may be determined for a given reinforced embankment. In this case, the procedure involves 4 steps.

Step 1: Problem Definition

Define the required parameters and geometry of the problem as indicated in Figure 5.7.

Design Parameters:

(a) Geometry:	h, D, B, t
(b) Fill Parameters:	c_f, ϕ_f, γ_f
(c) Foundation Parameters:	c_U, E_U, γ
(d) Geotextile Parameters:	E_f, T

To illustrate the procedure, consider the final design arrived at in Example 1, including the same design parameters. We are required to estimate the factor of safety, F, of this embankment against collapse.

Steps 2 through 4 are carried out identically to Example 1.

The factor of safety, F, is equal to the critical factor of safety, F_c, calculated in Step 4. (Step 5 is unnecessary.)

For this example, the factor of safety, F, is 1.3 and the design is governed by the allowable compatible strain.

Comparison of Limit Equilibrium and Finite Element Results

The proposed extended limit equilibrium method was used to estimate collapse heights for the various cases analysed with the finite element method. A comparison of the collapse heights for reinforced embankments obtained using the limit equilibrium technique with those deduced from finite element analyses indicated excellent agreement, with a maximum discrepancy of less than 4%.

5.7/ Construction Considerations

The design recommendations given in the previous section should result in satisfactory embankment performance (recognizing that large settlements may occur for embankments on peat) provided that a reasonable construction procedure is adopted. In particular, the following points, which are primarily for construction over peat deposits, should be considered:

1) The geotextile should be placed as close as practical to the original ground surface. If the surface will support workmen and is close cut, the geotextile should be stretched out on the surface directly ahead of the fill placement. Local depressions which cannot reasonably be bridged with the geotextile should, where possible, be filled with lightweight fill. (Filling of depressions with conventional fill prior to placing the geotextile may result in excessive settlement or even local shear failure.)

2) If a working mat must be placed, it should, if possible, be constructed on lightweight fill. The thickness of the working mat should be kept to a minimum by using only lightweight equipment.

3) If the fill material contains a significant proportion of silt or clay size particles, then care must be taken to provide drainage at the foundation-fill interface. This drainage may take the form of a suitable granular filter and/or a filter fabric. A filter fabric should not be placed in contact with a reinforcing geotextile.

4) Care should be taken not to disturb or damage the root mat.

5) Rockfill is likely to cause local overstressing of any geotextile and will often damage the root mat. A geotextile selected in accordance with the procedures described elsewhere in this chapter may not be adequate if used in conjunction with rockfill.

6) In general, the maximum strain developed in the geotextile will be in the direction transverse to the embankment alignment. The geotextile should be placed with the stronger direction transverse to the alignment. Seams are to be avoided in the transverse direction. If a seam is present, it will generally control the geotextile performance.

7) Embankments on peat should be constructed such that the maximum excess pore pressure does not exceed 0.34 times the change in vertical stress due to embankment construction. If in doubt, piezometers should be installed and the construction rate should be controlled based on these observations. In these cases, an initial estimate of an allowable construction rate would be 0.9 m of fill per week. This rate should be adjusted in light of observed field performance.

5.8/ Future Field Instrumentation and Testing

Of necessity, the recommendations made in this report have been based on limited field observations. It is considered essential that additional instrumentation and monitoring of reinforced embankments be undertaken and that these data be used to corroborate or modify the recommendations presented in this report. The instrumentation should monitor the settlement of the embankment and, when possible, the strains in the geotextile. Appendix C contains some discussion of factors to be considered in the field instrumentation.

It should be emphasized that field instrumentation and monitoring will be of little value unless it includes a record of the construction sequence and the excess pore pressures induced during construction, as well as a detailed field investigation of the underlying soils. Any instrumentation of an embankment on peat should be accompanied by appropriate site investigation and laboratory testing as discussed in Sections 6.4 and 6.5. In particular, samples of peat should be obtained using 51 mm diameter Shelby tubes and a 150 mm diameter piston sampler (available from U.W.O.). Testing on the peat should include moisture and ash content determination, 1-D consolidation tests, and either ring shear or simple shear tests (apparatus for performing simple shear tests are available at U.W.O.). The observed performance of the embankment should then be related to the compressibility and shear strength of the peat and to the excess pore pressures induced during construction.

6/ Conclusion

This report provides guidelines for the selection and use of geotextiles as reinforcement in low embankments on soft/weak foundations.

The parameters necessary for the design of reinforced embankments on soft/weak soils are described and the techniques for determining these parameters discussed. Recommendations for specific tests are made.

It is shown that the design and method of construction can greatly affect embankment performance and consideration should be given to the design and construction guidelines if shear deformations are to be limited and collapse avoided.

Finally, it is recommended that additional field instrumentation of reinforced embankments on soft/weak foundations be carried out in conjunction with appropriate laboratory testing so as to confirm the applicability of the recommendations for soils other than those specifically examined in this study.

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Appendices

Editor's Note: — The appendices have not been included with this report because they are rather bulky and of limited interest. However, those who wish to view them may do so at the MTC Head Office in Downsview. Contact the Materials Research Office, Research and Development Branch.

A brief outline of each appendix is provided below:

Appendix A/

The fabric reinforced test section built at Pinto Pass, Mobile, Alabama, has played a major role in the development of data and concepts for use in the design of geotextile reinforced embankments. In this appendix, a finite element soil-structure interaction analysis is used to examine the behaviour of the test section so as to provide additional insight into the role of the geotextile.

Appendix B/

Two test embankments were constructed in 1979 on a soft, thick clay-peat deposit at Almere, Germany, and reported in 1981. The appendix compares the results of a detailed finite element analysis with both field observations and the results of simplified analyses. The implications for design using geotextiles are discussed.

Appendix C/

The Ontario MTC designed and constructed a portion of a road embankment in 1981/82 on a soft organic deposit using geotextiles as reinforcement. A number of arbitrary assumptions were made and the design of the embankment is described, together with the basic engineering properties of the peat and the results of field measurements.

Appendix D/

MTC constructed a geotextile reinforced embankment on a highly compressible deposit at the Bloomington Road near Aurora, Ontario. Limit equilibrium techniques were used. The appendix describes the engineering properties required for a detailed finite element analysis of the embankment behaviour at two instrumented test sections. The results of these analyses are compared with the observed field behaviour. The implications of this case history for future design using geotextiles are discussed.

Appendix E/

This appendix summarizes a preliminary investigation into the determination of engineering parameters for peat. The peat was obtained from a field investigation in the vicinity of Bloomington Road, near Aurora, close to the two previous investigations by MTC described in Appendix C.

Appendix F/

In order to determine the strength and deformation behaviour of the soil-fabric interface, a direct shear apparatus was constructed which meets a number of predetermined criteria. It should be noted that, due to the design assumptions, there are limitations to the use of the direct shear apparatus. The properties that can be determined from direct shear tests are outlined in this appendix. The teat results obtained for seven geotextiles tested in the two fill materials are presented.

Appendix G/

The apparatus, test methods, and test results relating to use of the wide strip method for determining the stress-strain characteristics of seven geotextiles are presented.

Appendix H/

The tensile testing method used in the wide strip method procedure operates on the principle of a constant-rate-of-specimen-extension. This method was adopted because the stress-strain characteristics of many fabrics are rate-dependent. The basic distinction between this and other methods for measuring strip tensile properties is the width of the specimen.